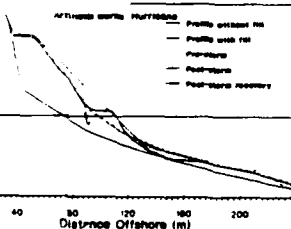
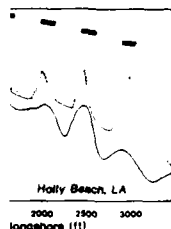
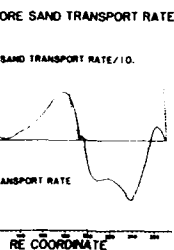




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# SHORELINE CHANGE AND STORM-INDUCED BEACH EROSION MODELING: A COLLECTION OF SEVEN PAPERS

Edited by

Nicholas C. Kraus

Coastal Engineering Research Center

DEPARTMENT OF THE ARMY  
Waterways Experiment Station, Corps of Engineers  
3909 Halls Ferry Road, Vicksburg, Mississippi 39180-6199



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Under Shoreline and Beach Topography Response Modeling  
Work Unit 32592 and Calculation of Cross-Shore Sediment  
Transport and Beach Profile Change Work Unit 32530

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one appears in the Proceedings of the Beach Technology '88 conference. Coastal Zone '89 was held under the auspices of the American Society of Civil Engineers, and Beach Technology '88 was held under the auspices of the Florida Shore and Beach Preservation Association. In support of the Coastal Zone '89 conference, the editor of this report organized a special session of five of the papers included here under the session theme, "Shoreline Change and Storm-Induced Beach Erosion Modeling," also used as the title of this report.

This information is expected to be of interest to US Army Corps of Engineers field offices and other public and private organizations involved with technical aspects of beach change modeling and the use of models in project planning and design.

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## PREFACE

Portions of the work described herein were authorized as a part of the Civil Works Research and Development Program by Headquarters, US Army Corps of Engineers (HQUSACE). Work was performed under the Shoreline and Beach Topography Response Modeling Work Unit 32592, and the Calculation of Cross-Shore Sediment Transport and Beach Profile Change Work Unit 32530 which are part of the Shore Protection and Restoration Program at the Coastal Engineering Research Center (CERC), US Army Engineer Waterways Experiment Station (WES). Messrs. John H. Lockhart, Jr., James E. Crews, and John G. Housley were HQUSACE Technical Monitors. Dr. Charles L. Vincent was Program Manager for the Shore Protection and Restoration Program at CERC.

The studies at CERC were performed over the period 1 January 1988 through 30 October 1989 by Dr. Nicholas C. Kraus, Senior Scientist, Research Division (RD), CERC; Dr. Norman W. Scheffner, Research Hydraulic Engineer, and Mr. Mark G. Gravens, Hydraulic Engineer, Coastal Processes Branch (CPB), RD; and Dr. Steven A. Hughes, Hydraulic Engineer, Wave Dynamics Division (WDD), CERC. Collaborators in this work were Drs. Hans Hanson and Magnus Larson, Department of Water Resources Engineering, Institute of Science and Technology, University of Lund, Sweden, and Dr. Lindsay Nakashima, formerly of the Coastal Geology Section, Louisiana Geological Survey, and presently at Woodward-Clyde Consultants, Baton Rouge, Louisiana. Acknowledgments for site-specific studies are contained within the main text.

The studies at CERC were under general administrative supervision of Dr. James R. Houston and Mr. Charles C. Calhoun, Jr., Chief and Assistant Chief, CERC, respectively, and under direct administrative supervision of Mr. H. Lee Butler, Chief, RD; Mr. Claude E. Chatham, Jr., Chief, WDD; and Mr. Bruce A. Ebersole, Chief, CPB.

Dr. Kraus coordinated development and review of the papers. Mr. Gravens and Dr. Mark R. Byrnes, CPB, were Principal Investigators of Work Units 32592 and 32530, respectively. Ms. Carolyn J. Dickson, CPB, reformatted the papers and provided organizational support in preparing the manuscript.

COL Larry B. Fulton, EN, was Commander and Director of WES during final report preparation. Dr. Robert W. Whalin was Technical Director.

## FOREWORD

This report consists of seven papers dealing with prediction of beach change by means of numerical simulation models. The papers were recently published by members of the Coastal Engineering Research Center (CERC), US Army Engineer Waterways Experiment Station (WES), and colleagues from other organizations. The papers collectively provide an overview of the state of research and engineering capabilities of numerical modeling of beach change, as well as a framework for understanding the role of modeling in planning and design of shore protection projects. This information is expected to be of interest to US Army Corps of Engineers field offices and other public and private organizations involved with technical aspects of beach change modeling and the use of models in project planning and design.

Each paper comprises a chapter of this report. Five of the papers appear in the Proceedings of the Coastal Zone '89 conference, one is an updated and expanded version of a paper appearing in that Proceedings, and one appears in the Proceedings of the Beach Technology '88 conference. Coastal Zone '89 was held under the auspices of the American Society of Civil Engineers, and Beach Technology '88 was held under the auspices of the Florida Shore and Beach Preservation Association. In support of the Coastal Zone '89 conference, the editor of this report organized a special session of five of the papers included here under the session theme, "Shoreline Change and Storm-Induced Beach Erosion Modeling," also used as the title of this report.

Six of the papers were reformatted and minor corrections made in phraseology for publication in this report. The reformatted versions can be considered as reprints of the originals which appear in the conference Proceedings, and the citation to the source is given at the top of the respective title page. The paper by Mark B. Gravens is a substantially revised version of his paper appearing in the Proceedings of Coastal Zone '89 and includes final results and conclusions not available at the time of writing of the conference paper. Therefore, it is an original contribution.

The papers treat three major topics; use of numerical simulation models in project planning and design, prediction of long-term shoreline change, and prediction of the response of the beach profile to storms. The first two

papers primarily concern modeling and the planning process. The paper by Nicholas C. Kraus develops a general framework for understanding the role of numerical models of beach change in the planning and design process for shore protection, and it also serves as an introduction to the technical papers which follow. The paper by Steven A. Hughes describes an actual project and the application of various types of models, illustrating some of the principles described in the preceding paper.

The five remaining papers treat technical aspects of numerical simulation of beach change, emphasizing procedures and results rather than mathematical details. In development of the technical papers, an effort was made to present the state of the art in both research and application of the models. The paper by Hans Hanson and Nicholas C. Kraus presents the first description of a recent advance in shoreline change modeling, the capability to describe shoreline change produced by detached breakwaters that transmit wave energy, and it includes tests of the model and verification for Holly Beach, Louisiana. The paper by Mark B. Gravens describes an intensive application of the shoreline change model to investigate the effect of construction of a proposed entrance channel on the beach at Bolsa Chica, California. The shoreline change project at Bolsa Chica is put in a broader perspective of a multitasked study in the paper by Steven A. Hughes.

The final three papers concern modeling of storm-induced beach erosion. The two papers written by Magnus Larson and Nicholas C. Kraus describe tests of a newly developed model of storm-induced beach and dune erosion which has some capability to simulate beach recovery after storms. They apply the model to examine the relative behavior of two generic types of beach-fill cross-sections for protection against attack by hypothetical storms and also discuss the methodology of applying this emerging technology. In the third paper on storm erosion, Norman W. Scheffner summarizes an application of a model of storm-induced beach erosion to the north New Jersey coast. He takes a statistical approach by which dune erosion-frequency of occurrence curves are developed by driving the model with waves and water levels available from a large data base encompassing both hurricanes and northeasters.

Nicholas C. Kraus  
Editor

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## BEACH CHANGE MODELING AND THE COASTAL PLANNING PROCESS

Nicholas C. Kraus<sup>1</sup>

### ABSTRACT

This paper describes the role of beach change numerical modeling in the process of planning, design, and evaluation of shore protection projects. Topics discussed include the capabilities of models, selection of the appropriate model, applications of models to coastal planning, and how coastal managers can create conditions which will maximize returns from models and lead to improved predictions of project performance. The paper also serves as a general introduction to more detailed papers on model applications given in a special session of the Coastal Zone '89 conference entitled "Shoreline Change and Storm-Induced Erosion Modeling."

### INTRODUCTION

Beach stabilization and coastal flood protection are two major areas of concern in the field of coastal engineering. Erosion, accretion, and change in offshore bottom topography occur naturally through the transport of sediment by waves and currents. Additional changes result from perturbations introduced by coastal structures, beach fills, and other engineering activities. Beach change is controlled by wind, waves, currents, water level, nature of the sediment and its supply, and constraints on sediment movement, such as those imposed by coastal structures. These sediment processes are nonlinear and have great variability in space and time. Although it is a challenging problem to predict the course of beach change, such estimations are necessary to design and maintain shore protection projects.

Prediction of beach evolution with numerical models has proven to be a powerful technique that can be applied to assist in the determination of project design. Models provide a framework for developing project problem formulation and solution statements, for organizing data collection and

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analysis and, importantly, for efficiently evaluating alternative designs and optimizing the selected design. Most of the physical factors mentioned above and their interaction can be represented in numerical simulation models.

This paper describes the use of numerical models in the planning process for shore protection. It also introduces general concepts and capabilities expanded upon in companion papers (Gravens 1989, Hanson, Kraus, and Nakashima 1989, Scheffner 1989, Larson and Kraus 1989) on models given in a special session of the Coastal Zone '89 conference entitled "Shoreline Change and Storm-Induced Erosion Modeling."

## **TYPES OF MODELS**

### **Coastal Experience / Empirical Models**

The best "model" is to know the optimal project design from experience. Because of the complexity of beach change, design decisions should be grounded on "empirical modeling," i.e., adaptation and extrapolation from other projects on coasts similar to the target site. Coastal experience and understanding of coastal processes (waves, currents, sediment transport) and geomorphology are essential. However, prediction through coastal experience without the support of an objective, quantitative tool, such as a numerical model, has limitations:

- a. It relies on the judgment of specialists familiar with specific regions of the coast and on experience with previous projects, which may be limited, inapplicable, or anachronistic.
- b. It is subjective and does not readily allow comparison of alternative designs with quantifiable evaluations of relative advantages and disadvantages. Also, conflicting opinions can lead to confusion and ambiguity.
- c. It is not systematic in that it may not include all pertinent factors in an equitable manner.
- d. It does not allow for estimation of the functioning of new, novel, or complex designs. This is particularly true if the project is built in stages separated by long time intervals.
- e. It cannot account for the time history of sand transport as produced, for example, by variations in wave climate, modifications to coastal structures, and modification of the beach.
- f. It does not provide a methodology and criteria to optimize project design.

Finally, complete reliance on coastal experience places full responsibility of project decisions on the judgment of the engineer and planner without recourse to external and alternative procedures.

#### **Beach Change Numerical Models**

The capabilities of the various types of beach change numerical models are compared in this section. Fig. 1 extends and updates the classification scheme of Kraus (1983) for comparing models of beach evolution by their spatial and temporal domains of applicability. The domains were estimated by consideration of model characteristics, accuracy, and computation costs. The ranges of these domains will expand as knowledge of coastal sediment processes improves, models are improved and refined, wave and beach topography data become more abundant, numerical schemes become optimized, and computer costs decrease. The remainder of this section will discuss the capabilities and limitations of the classes of models compared in Fig. 1.

#### **Analytical models of shoreline change**

Analytical models are closed-form mathematical solutions of a simplified differential equation for shoreline change derived under assumptions of steady wave conditions, idealized initial shoreline and structure positions, and simplified boundary conditions. Longshore sand transport is represented, whereas cross-shore transport is omitted, yielding a 1-dimensional (1D) model. Because of the many simplifications needed to obtain closed-form solutions, particularly the assumption of constant waves, analytical models are usually too crude for use in design. Analytical solutions serve as a means to examine trends in shoreline change and to investigate basic dependencies of the change on waves and initial and boundary conditions. Larson, Hanson, and Kraus (1987) give a survey of more than 25 new and previously derived analytical solutions of the shoreline change equation.

#### **Profile change / beach erosion models**

Beach erosion models calculate sand loss on the upper profile resulting from storm surge and waves (Kriebel 1982, Kriebel and Dean 1985, Larson 1988, Scheffner 1988, 1989). This 1D model is simplified by omitting longshore sand transport processes, i.e., constancy in longshore processes is assumed, so that only one profile at a time along the coast is treated. Although such

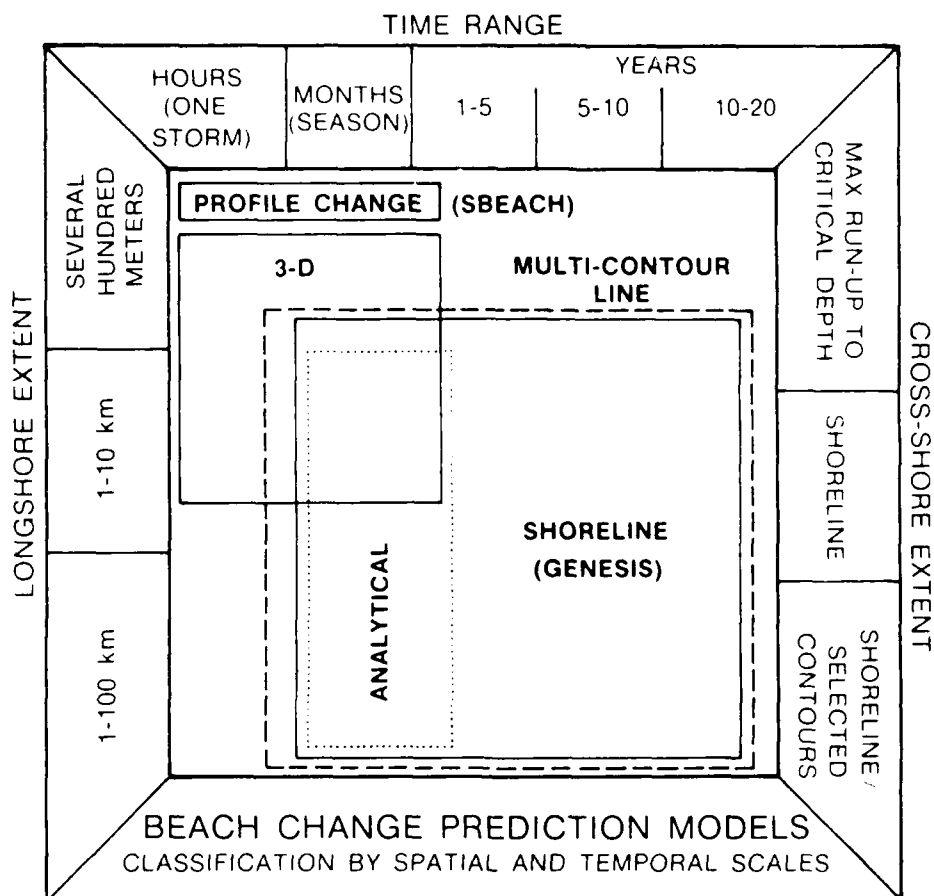


Fig. 1. Classification of beach change models

models can calculate with some reliability beach erosion produced by large storms, considerable research remains to be done to extend them to simulate major morphological features of the profile, such as bars and berms, and beach recovery (Larson 1988, Larson, Kraus, and Sunamura 1988, Kraus and Larson 1988, Larson and Kraus 1989) and hence become true "profile change" models.

**Shoreline change model**

The shoreline change numerical model is a generalization of analytical shoreline change models. This 1D model enables calculation of the shoreline response to wave action under a wide range of beach, coastal structure, wave, and initial and boundary conditions, and these conditions can vary in space

and time (Kraus 1983, Kraus and Harikai 1983, Kraus, Hanson, and Harikai 1984, Hanson and Kraus 1986a, Hanson 1987, Hanson and Kraus 1989, Gravens and Kraus 1989). Despite the assumption of constancy of beach profile shape alongshore, the shoreline change model has proven to be robust in predictions and provides a complete solution of the equation governing shoreline change. Because the profile shape is assumed to remain constant, in principle, onshore and off-shore movement of any contour could be used to represent beach change. Thus, this type of model is sometimes referred to as a "one-contour line" model or, simply, "one-line" model. Since the mean shoreline position (zero-depth contour) is conveniently measured and such data are usually available, the representative contour line is taken to be the shoreline.

#### Multi-contour line / schematic three-dimensional (3D) models

Three-dimensional beach change models describe the response of the bottom to waves and currents, which can vary in both horizontal (cross-shore and longshore) directions. Therefore, the fundamental assumptions of constant profile shape used in shoreline change models and constant longshore transport in beach change models are relaxed. Although 3D models are the ultimate goal of deterministic calculation of sediment transport and beach change, achievement of this goal is limited by our capability to predict sediment transport processes and wave climates. In practice, simplifying assumptions are made to produce schematic 3D-models, for example, to restrict the shape of the profile or calculate global rather than point transport rates. Perlin and Dean (1978) introduced an extended version of the "2-contour line model" of Bakker (1968) to an n-contour line model in which depths were restricted to monotonically increase with distance offshore.

Schematized 3D beach change models have not yet reached the stage of wide application; they are limited in capabilities due to their complexity and require considerable computational resources and expertise to operate. Introduction of these models into engineering practice is expected in the near future, however.

### Fully 3D models

Fully 3D-beach change models represent the state-of-art of research. Waves, currents, sediment transport, and changes in bottom elevation are calculated point by point in small areas defined by a horizontal grid placed over the region of interest. Use of these models requires special expertise, powerful computers, and extensive field data collection programs (Vemulakonda et al. 1988), and applications have been limited to large and high-funded projects. Because fully 3D-beach change models involve the detailed physics of sediment transport, they require extensive verification and sensitivity analyses.

### Summary of model capabilities

Only two types of well tested beach change numerical simulation models are presently available for general use, namely, the storm-induced beach erosion model and the shoreline change model. The storm erosion model is site specific in that local profile information and storm statistics are the main inputs. This type of model is discussed in a deterministic approach by Larson and Kraus (1989) and in a statistical approach by Scheffner (1989) in papers companion to this one.

The shoreline change model requires comprehensive data on the local and regional levels. Therefore, it is an ideal vehicle for systemizing the planning process for coastal protection, and the remainder of this paper will deal with this model. Examples illustrating shoreline change model capabilities are given in companion papers by Gravens (1989) and Hanson, Kraus, and Nakashima (1989), and Hughes (1989).

The shoreline change numerical model simulates long-term evolution of the beach plan shape and provides a framework to perform a time-dependent sediment budget analysis. As such, its operation and output are readily understood by coastal engineers and managers. The model is robust in that it can describe a wide range of conditions encountered in shore protection projects. The Coastal Engineering Research Center of the U.S. Army Engineer Waterways Experiment Station is in the final stages of releasing the model GENESIS (GENEralized model for SIMulating SHoreline change) (Hanson 1987, 1989, Hanson and Kraus 1989) for widespread use in the Corps of Engineers. Much of the

material described in this paper was gained by experience in applying GENESIS and its predecessor model on numerous projects.

## SHORELINE CHANGE MODEL

### Uses

The shoreline change model is best suited to situations in which a systematic trend exists in long-term change of shoreline position, such as retreat downdrift of a groin or jetty, and advance of the shoreline behind a detached breakwater. The dominant cause of shoreline change in the model is related to changes in the sand transport rate along the coast produced by waves and wave-induced currents. Cross-shore transport processes such as storm-induced erosion and cyclical movement of the shoreline produced by seasonal variations in wave climate are assumed to cancel or to average out over a long simulation period.

Figs. 2a-c show an example of shoreline change which is well suited for modeling (Kraus and Harikai 1983, Kraus, Hanson, and Harikai 1984). The site is Oarai Beach, located about 180 km north of Tokyo on the Pacific Ocean coast of Japan. A 500-m long groin was constructed to protect a fishing harbor from infiltration by sand carried by the longshore current (long groin located at  $x = 0$  in Fig. 2). Figs. 2a and 2b show that the shoreline had a clear tendency to advance on the updrift side of the long groin independent of season if the interval between compared surveys is one year. Fig. 2c gives a plot of shoreline positions surveyed during each season of one year. The tendency of the shoreline to advance is partially obscured because the relatively short interval of 3 months includes the effect of individual storms and other seasonal variations in wave climate, such as changes in predominant direction and wave steepness, on shoreline position.

### Duration of Simulation

The duration of the simulation depends on the wave and sand transport conditions, characteristics of the project, and whether the beach is close to or far from equilibrium. Immediately after completion of a project, the beach is far from equilibrium, and changes resulting from longshore sand transport dominate over storm and seasonal changes. Shoreline change calculated over a short interval will probably be reliable in such a case. As the beach approaches equilibrium with the project, the simulation interval must usually be

extended to a number of years to obtain valid predictions. Stated differently, the shoreline change model best calculates shoreline response in transition from one equilibrium state to another, which occur over months to years.

#### **Spatial Extent of Simulation**

The spatial extent of a region to be simulated with a shoreline change model can range from the single project scale of hundreds of meters to the regional scale of tens of kilometers. The modeled longshore extent will mainly depend on the physical dimensions of the project and boundary conditions controlling the sand transport. Dimensions of the project are at a local level, whereas placement of boundary conditions may or may not require extension to a regional level. Evaluation of possible effects of the project on neighboring beaches may also dictate extension of the spatial range of the simulation. Shoreline change numerical models require minimal computer resources and are usually capable of covering a regional scale for engineering studies.

As previously discussed, shoreline change models are designed to describe long-term trends of the beach plan shape in the course of its approach to an equilibrium form. This change is usually caused by a notable perturbation (for example, construction of a groin or jetty). Shoreline change models are not applicable to simulating a highly fluctuating beach system in which no trend in shoreline position is evident, such as on a long natural beach. Specifically, the shoreline change model GENESIS, in its present form (Version 2), is not applicable to calculating beach change in the following situations: interior of inlets or areas dominated by tidal flow; storm-induced beach erosion in which cross-shore sediment transport processes are dominant; scour at structures; and sediment transport processes in the offshore.

#### **Capabilities**

Table 1 gives a summary of major capabilities and limitations of Version 2.0 of the shoreline change simulation model GENESIS.



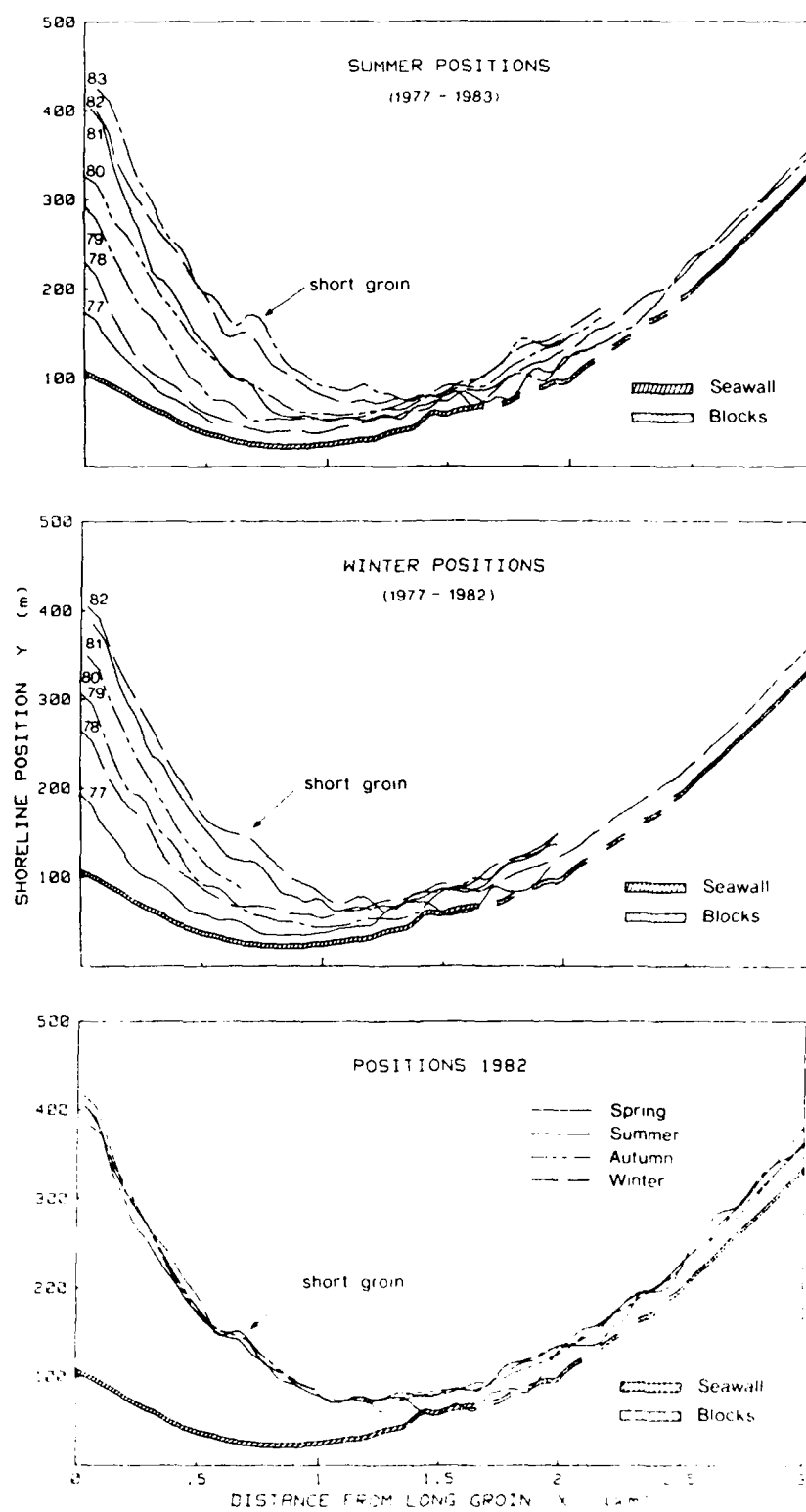


Fig. 2. Shoreline change measured in the vicinity of a long groin  
 (a) summer positions, (b) winter positions, (c) seasonal positions

Table 1

Capabilities and Limitations of GENESIS Version 2.0

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Capabilities

- \* Almost arbitrary numbers of groins, jetties, detached breakwaters, beach fills, and seawalls
- \* Structures and beach fills in almost any combination
- \* Compound structures such as T-shaped groins and spur groins
- \* Bypassing of sand around and transmission through groins and jetties
- \* Diffraction at detached breakwaters, jetties, and groins
- \* Wave transmission through detached breakwaters
- \* Coverage of wide spatial extent
- \* Offshore input waves of arbitrary height, period, and direction
- \* Multiple wave trains (as from independent wave sources)
- \* Sand transport produced by oblique wave incidence and by alongshore gradient in wave height
- \* Highly automated, numerically stable, and well tested

Limitations

- \* No wave reflection from structures
- \* No tombolo development in a strict sense (shoreline not allowed to touch a detached breakwater)
- \* Slight restrictions on location, shape, and orientation of structures
- \* Basic limitations of shoreline change modeling theory<sup>1)</sup>

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1) Note: For further information on the theory of shoreline change numerical modeling and GENESIS, see Hanson and Kraus (1989)

## SHORELINE CHANGE MODELING AS A TOOL IN THE PLANNING PROCESS

### Elements of the Planning Process

This section discusses the role of shoreline change numerical modeling in the overall process of planning, design, construction, and evaluation of project performance. The material addresses the question of how a shoreline change model fits in the decision process of coastal management. The purpose of such planning is to determine the most effective socio-economic engineering solution to a shore protection problem. The planning process consists of the following steps:

- a. Formulate problem statement, identify constraints, and develop criteria for judging the performance of the project.
- b. Assemble and analyze relevant data.
- c. Determine project alternatives.
- d. Evaluate alternatives. (Return to Step a, as necessary)
- e. Select and optimize project design.
- f. Construct the project.
- g. Monitor the project.
- h. Evaluate the project according to Step a and report the results.

These steps and their interrelation are shown diagrammatically in Fig. 3, in which stages in the planning process where modeling can take an active role are designated with the word "model" in parentheses.

Step a. A clear problem statement and criteria for judging the project design (including the advantages/disadvantages of design alternatives) must be developed to determine in an objective manner the success or failure of the project. The problem statement and judgment criteria should be explicit. Otherwise, passage of time between project planning and the performance evaluation may obscure the original purpose, and project functioning may be evaluated out of context.

The problem statement and judgment criteria will usually encompass several factors, including local and regional considerations. This is called comprehensive planning, as opposed to single-project planning. For example, suppose a section of road along a coast is threatened by erosion. One possible problem statement is that erosion is endangering major resources between

points A and B. A criterion for judging the solution would be to halt the erosion for less than X dollars in initial construction and less than Y dollars in annual maintenance. Suppose that a revetment is selected as providing the optimal solution and is constructed and maintained within budget. Also, monitoring shows that the project performed as intended. The project has satisfied the original objectives under single-project planning. However, if, after construction, it is determined that the beach downdrift of the project had eroded because of sand deprivation (caused, for example, by encasement of sand by the revetment), it may be judged that the project was a failure. A similar project might have as its comprehensive planning problem statement protection of the road and mitigation of erosion of the downdrift beach. This would lead to a different solution, for example, a revetment to protect the road and periodic nourishment for the downdrift beach.

It is essential to distinguish failures in planning and failures in projects themselves if lessons are to be learned from experience.

Step b. All relevant data should be assembled and analyzed with a view toward both defining the problem statement and arriving at a solution approach. In the example given above, an evaluation of data on shoreline change and the predominant direction of longshore sand transport would have led to a more comprehensive problem statement. Data gaps, such as lack of shoreline position data and wave data, may suggest establishment of data collection programs and wave hindcasts.

Steps c and d. Development of a project from the point of identification of the problem through construction and performance evaluation involves consideration of five general criteria:

- (1) Technical feasibility.
- (2) Economic justification.
- (3) Political feasibility.
- (4) Social acceptability.
- (5) Legal permissibility.

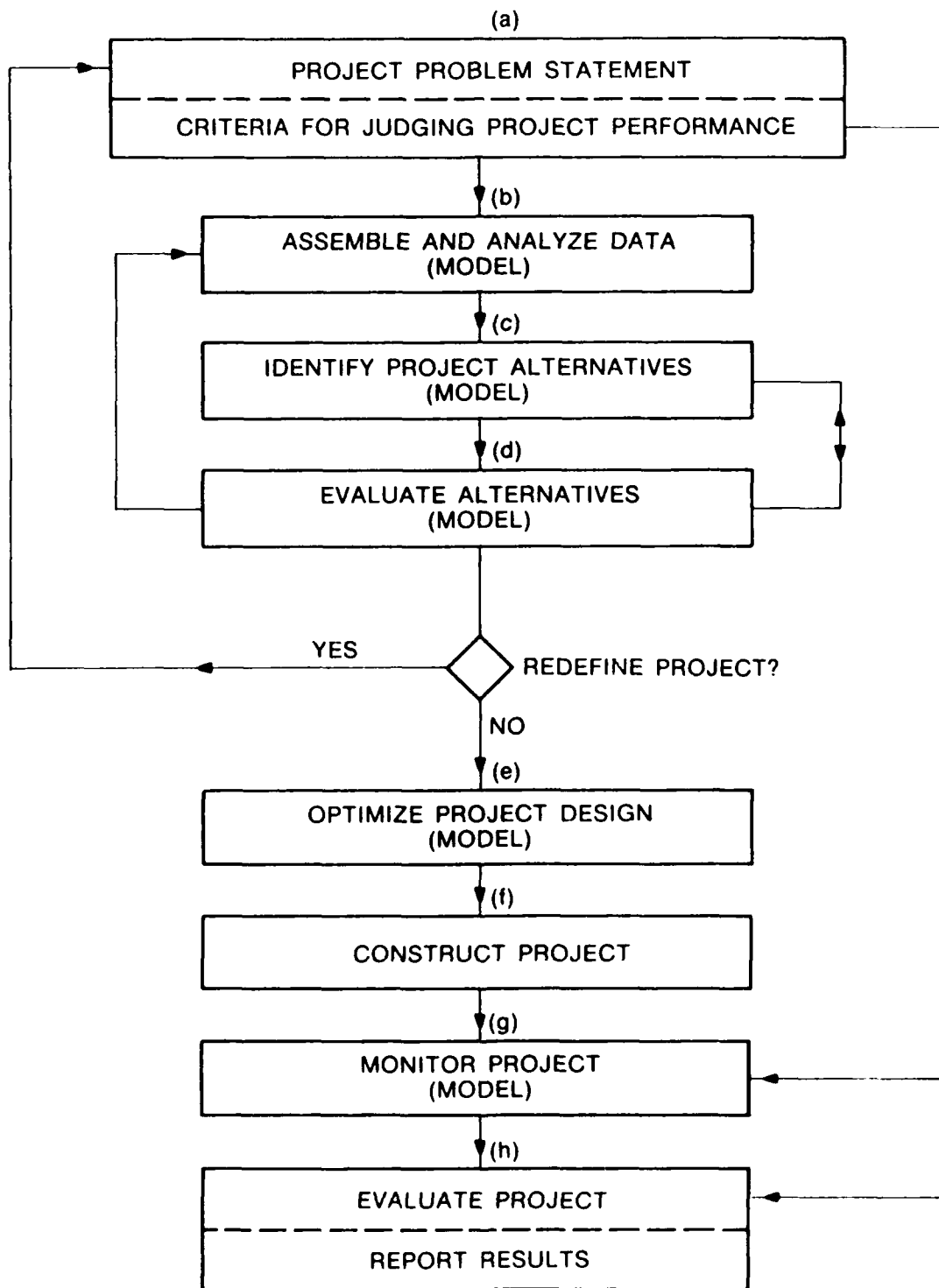


Fig. 3. Major steps in project planning and execution

Technical feasibility concerns the magnitude of the wave, current, and sediment transport processes; availability of construction materials; limitations on project design due to external factors; and limitations on access to the site; and capabilities of the project staff. Economic justification concerns the project benefits and is typically the major driving force of a shore protection project. Funding for the project planning and design staff, and construction, maintenance, and monitoring costs also enter into the economic justification, as well as potential benefits. Economic justification, political feasibility, social acceptability, and legal permissibility are interconnected, since the local, state, and Federal governments share in the funding and permitting of a project.

Evaluation of alternatives involves simultaneous assessments of technical and economic feasibility to arrive at a cost-beneficial design. During the detailed investigation of alternatives and use of the data base developed at Step b, it may become apparent that the original problem statement and judgment criteria for the project need to be refined. For example, project planning may be initiated to satisfy a local need, but later evolve beyond the primary (site-specific) problem to include impacts on a regional scale (comprehensive planning).

Step e. Once an alternative is selected, it is necessary to optimize the design so that the greatest benefit is obtained for the least cost.

Steps f and g. After the project is constructed, it should be monitored to ascertain that the final design was implemented and to evaluate its performance. The monitoring plan is devised to answer the question of whether the project achieved its purpose according to the criteria developed at Step a. By designing the monitoring program to address Step a, both a productive and economical monitoring plan can be developed. Results of the project should be published and the processed data archived for use in future assessments and to serve as guidance in other projects.

#### **Role of Shoreline Change Modeling**

Shoreline change numerical modeling is closely associated with and can greatly aid the planning process described in the preceding section. Planners and engineers can use the guidance given below to establish an approach conducive to optimal use of modeling capabilities.

Step b. Data requirements of the shoreline change model cover a wide range of coastal-process and project-related information, as summarized in Table 2. Within the framework of shoreline change modeling, guidelines are available for collecting, reducing, and analyzing the data in a systematic manner. Most physical data needed for evaluating and interpreting shoreline and beach evolution processes in a wide sense are used in the shoreline change modeling methodology. Certain other data may be lacking in particular applications having unique requirements, so that coastal experience and overall project planning should not be subverted by complete dependence on shoreline change modeling requirements. For example, geological and regional factors may be involved, as through earthquakes, subsidence, or structure of the sea bottom substrata. Environmental factors such as water circulation and quality (temperature, salinity, sediment concentration, etc.), as well as biological factors should be considered. Thus, although a shoreline model such as GENESIS can simulate the movement of beach fill material placed at arbitrary locations and times along the beach, the breeding habits of sea turtles and birds may restrict the season and/or location of the fill. In summary, data requirements of the shoreline change model provide an organized and comprehensive first step in assembling the available data for project design.

Steps c-e. Shoreline change modeling provides a powerful tool for quantitative and systematic evaluation of alternatives and optimization of the final plan. As an example, Hanson and Kraus (1986b) simulated beach change for nine hypothetical combinations of plans to mitigate erosion at a recreational beach. The without-project ("do nothing") alternative and general shore protection schemes were evaluated for groins of various sizes and spacings, beach fills of various quantities, and a single, long detached breakwater. Technical criteria for judging the solution involved two factors, protection of the eroding beach and minimization of the quantity of sand transported downcoast which would enter the navigation channel of a fishing harbor. Shoreline change modeling readily allowed a matrix of shoreline change volumes to be compiled for target sections of the coast by which technical solutions could be ranked. Economic criteria were then applied to arrive at the most feasible project plan. In evaluation of Steps c-e, it may become apparent that other methodologies, such as physical modeling (for estimating wave

forces and overtopping, etc.), hydrodynamic modeling, and field data collection are needed. Hughes (1989) describes such an integrated, multi-tool approach.

Table 2  
Data Required for Shoreline Change Modeling

<u>Type of Data</u>	<u>Comments</u>
Shoreline position	Shoreline position at regularly spaced intervals along-shore by which the historic trend of beach change can be determined.
Offshore waves	Time series or statistical summaries of offshore wave height, period, and direction.
Beach profiles and bathymetry	Profiles to determine the average shape of the offshore beach. Bathymetry for transforming offshore wave data to values in the nearshore.
Structures and other engineering activities	Location, configuration, and construction schedule of engineering structures (groins, jetties, detached breakwaters, harbor and port breakwaters, seawalls, etc.). Structure porosity, reflection, and transmission. Location, volume, and schedule of beach fills, dredging, and sand mining. Sand bypassing rates at jetties and breakwaters.
Regional transport	Sediment budget; identification of littoral cells; location and functioning of inlets; river discharges; wind-blown sand.
Regional geology	Sources and sinks of sediment; sedimentary structure; grain size distribution (ambient and of beach fill); regional trends in shoreline movement; subsidence; sea level change.
Tide	Tidal range; tidal datum.
Extreme events	Large storms (waves, surge, beach erosion, failure of structures, etc.); inlet migration, opening, or closing; earthquakes.
Other	Wave shadowing by large land masses; strong coastal currents; ice; water runoff.



Step g. In addition to aiding in the evaluation and optimization of project designs, shoreline response modeling can provide guidance for preparing a monitoring plan (Step g). Regions of anticipated maximum and minimum shoreline change or sensitivity can be identified and the monitoring plan structured to provide data in these important regions. Initial estimates of the monitoring schedule (frequency of measurements) and density or spacing of measurement points can also be made by reference to the temporal characteristics of model predictions.

#### CONCLUDING DISCUSSION

Numerical models of beach change, particularly of profile erosion and shoreline change, are becoming more accurate and prolific, and they will be increasingly used in the planning and design process for shore protection. Because of their great power and generality, numerical models provide a framework for developing shore protection problem and solution statements, for organizing the collection and analysis of data and, most importantly, for evaluating alternative designs and optimizing the selected design. Mathematical models of beach evolution extend the coastal experience of specialists and introduce a systematic and comprehensive project management methodology to the local engineering or planning office.

This paper has attempted to demonstrate the utility and benefits of numerical modeling of coastal processes to the coastal planning and management community. Although emphasis was on numerical modeling and beach processes, it should be recognized that a shore protection project will involve a wide range of techniques and tools.

#### ACKNOWLEDGEMENTS

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## ENGINEERING ASSESSMENT OF PROPOSED BOLSA BAY DEVELOPMENT

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### ABSTRACT

The Waterways Experiment Station (WES) has examined the impacts that a proposed new ocean entrance and marina development at Bolsa Chica, California, would have on the ocean shoreline and tidal wetlands. This paper overviews the scope of the engineering studies, describes the engineering methodology applied by WES to examine possible impacts, and discusses products of the study. The emphasis of the paper is to illustrate how modern coastal engineering techniques can be used to aid coastal planners, developers, and government officials in making informed decisions about coastal resources.

### INTRODUCTION

The State of California, State Lands Commission (SLC), and others are reviewing a plan for a new ocean entrance system as part of a multi-use project. The project, located in the Bolsa Chica area of the County of Orange, California (Figure 1), includes navigational, commercial, recreational, and residential uses, along with increased flood protection and major wetlands restoration.

In order to satisfy requirements of the California Coastal Commission, which must "Confirm" the viability of a Land Use Plan it provisionally certified in January 1986, the SLC requested the US Army Engineer Waterways Experiment Station (WES) to conduct specific engineering studies regarding the technical and environmental assessment of a navigable and a non-navigable ocean entrance system at Bolsa Chica. Results of these studies will assist SLC (the principal public landowner in the project area) and other parties which are formulating reports and plans for the proposed Bolsa Bay project.

A joint effort involving WES's Coastal Engineering Research Center and Environmental Laboratory examined the impacts that the two proposed ocean entrance alternatives would have on the coastal shoreline and tidal wetlands.

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The studies assessed the impacts using both numerical and physical modeling techniques. Numerical models, using wave hindcasts developed at WES, were used to predict the long-term response of the adjacent shoreline resulting from construction of a jettied entrance. Numerical models were also used to estimate tidal flows and elevations within the proposed new wetlands area and transport and dispersion within the tidally-varying regions. From these results qualitative assessments of water quality were obtained. A 1-to-75 scale physical model of the proposed navigable ocean entrance system was constructed at WES to determine wave penetration into the marina area, to examine the influence of storm water flows into the complex, and to provide an initial functional configuration for the detached breakwater and entrance channel.

This paper reviews the purpose and scope of the WES studies, describes the engineering methodologies employed in the various phases of the effort, discusses representative products, and provides an overview of the studies so that nontechnical people involved in the Bolsa Chica decision process can obtain a more complete understanding of the role of the WES studies.

#### **PURPOSE AND SCOPE OF THE WES STUDIES**

Purpose. The primary purpose of the WES studies was to apply established engineering methodologies along with unique WES capabilities to estimate probable impacts that could result from the construction of either the proposed navigable entrance alternative, or the non-navigable entrance alternative at Bolsa Chica. In meeting this objective WES performed the following general tasks:

- a. Tested the proposed development concepts using both physical and numerical models.
- b. Analyzed and interpreted model results.
- c. Provided technical documentation of the study results.
- d. Presented study results at public workshops.

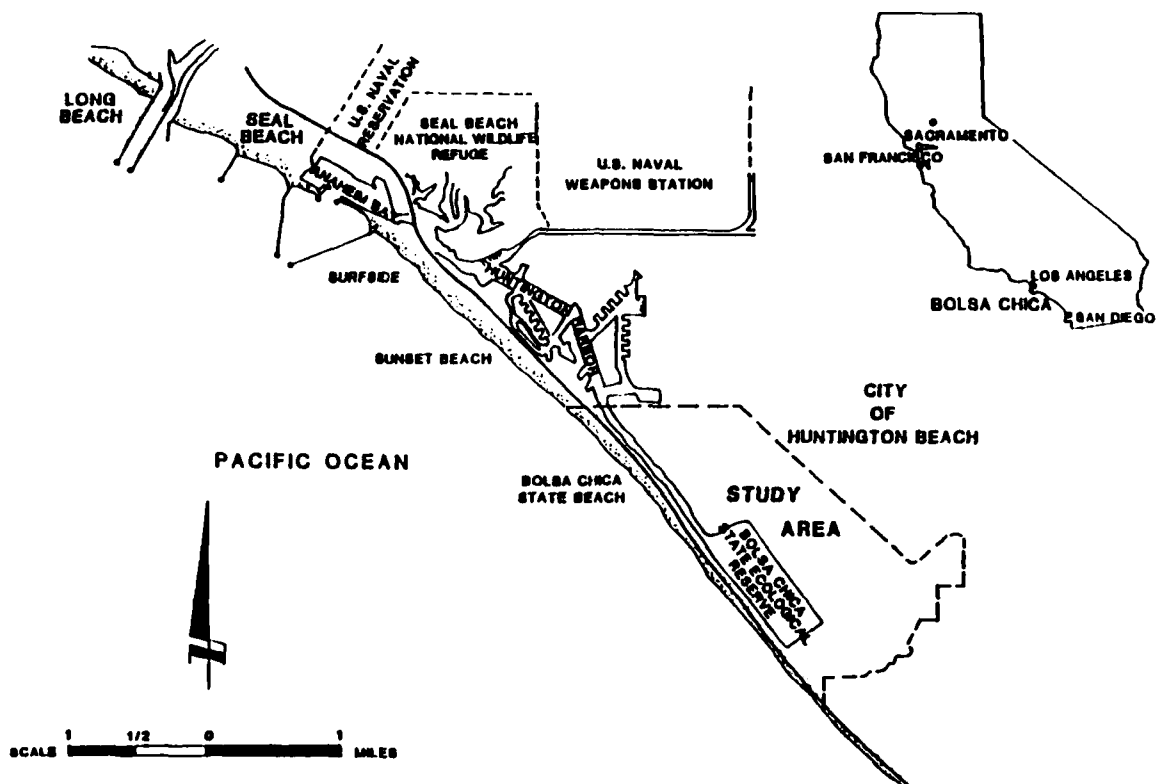


Figure 1. Bolsa chica study region location

It is also important to state what WES did not provide during the course of the studies. The following items were not part of WES's mission:

- a. WES did not provide project design. (Conceptual designs for testing were provided to WES by SLC. Design optimization will be performed by the private sector if a project is approved).
- b. WES did not (and does not) recommend one alternative over another. (Many more issues besides technical feasibility are involved in the Bolsa Chica decision process.)
- c. WES did not provide analysis of issues outside the WES scope of work.
- d. WES did not interpret study results in the context required for "Confirmation" hearings.

The Corps of Engineers, Los Angeles District (SPL), has also begun studies on Bolsa Chica known, as the "Feasibility Study", and it is important to establish the relationship between the WES studies and SPL's efforts. SPL's Feasibility Study will examine more alternatives than the two being examined by the WES studies, and SPL will consider more than just the technical issues being examined by WES.

Scope. The studies of the Bolsa Chica area conducted by WES are grouped into the following five general categories, three of which pertain to modeling:

- a. Numerical modeling of long-term shoreline response as influenced by placement of entrance channel stabilization structures, including sand management concepts.
- b. Numerical modeling of tidal circulation, including transport and dispersion of conservative tracers, in the Bolsa Bay, Huntington Harbour, and Anaheim Bay complex.
- c. Physical modeling of the proposed entrance channel, interior channels, and marina with regard to wave penetration, harbor oscillation, qualitative sediment movement paths, and storm water runoff.
- d. Assessment of the potential of the proposed non-navigable ocean entrance to maintain itself as a tidal inlet in an open configuration.
- e. Assessment of potential impacts to surfing that might arise from construction of a project at Bolsa Chica.

Details of these five tasks are provided in the following five sections.

#### SHORELINE RESPONSE NUMERICAL MODELING

Purpose. The purpose of the shoreline response modeling effort was to utilize a proven numerical shoreline simulation model to assess and quantify the potential long-term impacts of the proposed ocean entrance system at Bolsa Chica due to the longshore movement of beach sand, and to evaluate the potential for mitigation of any adverse effects induced by the entrance.

Tasks. The shoreline response modeling involved three major tasks: preliminary shoreline response modeling, 20-year wave hindcast of the Bolsa Chica region, and comprehensive shoreline response modeling. The preliminary modeling task utilized existing shoreline change data and existing wave data to develop, calibrate, and verify a shoreline change numerical model for the

project coast, extending from the Anaheim Bay east jetty downcoast to the mouth of the Santa Ana River. This task is termed preliminary because it estimates the range of potential impacts of a new entrance on adjacent beaches using the best wave data available at the time of the study. These preliminary estimates are of sufficient accuracy to determine the general range of impact. The wave hindcasting task was a 20-year numerical wave hindcast providing directional wave data at the Bolsa Chica project site for use in the comprehensive modeling task. The comprehensive shoreline response modeling task was similar to the preliminary modeling with the exception that hindcast waves were used as input to the shoreline response numerical model.

Methodology. The shoreline response model used in the Bolsa Chica studies is termed a "one-line" model. It assumes that the long-term planform shape of an open-ocean sandy coast is controlled by the incident waves and the longshore current they produce. Although it is recognized that other types of currents, as well as water level and wind also play a role in shoreline evolution, these processes are presumed to be secondary in the long term. Also, cross-shore transport is neglected under the assumption that the beach profile maintains an equilibrium form. Coastal improvements such as beach fills, jetties, breakwaters, and groins can be simulated in the numerical model. A complete description of this shoreline model is given by Hanson (1987) and Hanson and Kraus (1989).

The shoreline response numerical model: (a) takes an input specification for wave height, wave period, and wave direction at the seaward boundary; (b) refracts, diffracts, and shoals the waves over specified bathymetry to the break point; (c) calculates local longshore sediment transport rates at each longshore grid point; (d) determines the volume of sediment entering and leaving each shoreline grid cell; (e) updates the shoreline position based on net sand movement in or out of the cell; and (f) repeats the process with a new input wave condition at the boundary. For this study the offshore wave condition was updated at six-hour intervals for period of up to ten years.

Before the model can be applied to a specific site, it is necessary to supply the model with accurate nearshore bathymetry and to calibrate the model using historical shoreline movement data and representative wave climates for the region. Calibration consists of: (a) starting the model with a known



historic shoreline configuration; (b) inputting a time series of wave height, period, and direction at the model's seaward boundary; (c) running the model for a specified length of time; and (d) comparing the simulated changes to known historic shoreline changes. Depending on the quality of the comparison, the model can be adjusted by modification of two coefficients, and the calibration repeated until satisfactory reproduction is achieved.

After calibrating the model it is desirable to verify it by reproducing the shoreline change observed at the same site, but for a different time period than used for model calibration. No coefficient adjustment is made during the verification run. After verification, the model can be used to provide reasonable engineering estimates of future changes associated with suggested projects at the site.

Preliminary Shoreline Response Modeling. The following is a short overview of the preliminary shoreline modeling task. A complete description of this task and study results is given by Gravens (1988). The scope of work for this task included the following:

- a. Collection and review of existing wave and shoreline processes data along the project reach.
- b. Preparation and calibration of a shoreline response prediction model to estimate the adjacent shoreline impacts of the proposed navigable and non-navigable entrances.
- c. Identification and comparison of available wave data sources, selection of the most appropriate data source, and performing a nearshore wave transformation analysis.
- d. Calibration and verification of the shoreline response model using known quantities of beach evolution from surveyed shoreline positions.
- e. Application of the calibrated model to predict future shoreline changes resulting from construction of the navigable ocean entrance channel.

Shoreline change simulations covering a ten-year period over the reach of coast from Anaheim Entrance southward to the Santa Ana River were compared

Table 1. Preliminary Model Simulations

- 
1. 800-ft Channel, Proposed Site, No Sand Management
    - (a) Wave Heights Increased 15%
    - (b) Wave Heights Decreased 15%
    - (c) Wave Angles Shifted +10 Degrees
    - (d) Wave Angles Shifted -10 Degrees
  2. 1000-ft Channel, Proposed Site, No Sand Management
  3. 800-ft Channel, Warner Avenue, No Sand Management
  4. 800-ft Channel, South of Site, No Sand Management
  5. 800-ft Channel, Proposed Site, Dog-Leg, No Sand Management
  6. 800-ft Channel, Proposed Site, 7 Sand Management Concepts
  7. Simulated Shoreline Response Without Project
- 

for a variety of conditions, including a structured navigable entrance without sand management, a navigable entrance with sand management, and a no-project (existing condition) simulation.

Table 1 summarizes the simulations performed. The four variations performed during the first simulation demonstrated the model's sensitivity to input wave height and wave angle, and it also provided a probable range of shoreline impact. As expected, wave angle variations were more important. Shoreline response simulations calculated and plotted projected shoreline positions for 5- and 10-year time periods after construction of a project. These preliminary modeling efforts examined the following:

- a. Differences in shoreline impact due to channel width.
- b. The effect of locating the project upcoast or downcoast from the proposed location.
- c. The estimated annual net longshore transport rate at Bolsa Chica in comparison to historical estimates. (The comprehensive model will verify the range).
- d. The effect of continuing the present beach nourishment project at Sunset Beach.

Wave Information Study (WIS). Hindcasting of historical wave conditions on the Nation's coastline is an ongoing mission of the Corps of Engineers. Pacific coast wave hindcasts for the years 1956-1975 were beginning at the onset of the Bolsa Chica studies. With augmented funding through SLC, WES was able to complete a 20-year wave hindcast for the Bolsa Chica region so that results could be incorporated into this study. The purpose of the numerical wave hindcast effort was to provide reliable estimates of wave conditions occurring at the project site for use in the comprehensive shoreline response model and in the physical model of the proposed entrance channel.

The WIS hindcast starts with synoptic-scale pressure charts of the Pacific Ocean (in this case), and processes these data numerically to generate wind fields over the ocean basin. The winds are then input to a numerical wave prediction model that provides directional wave spectra at deepwater grid points along the coastline. Next, a spectral transformation numerical model propagates the deepwater waves into the shallow coastal waters, taking into account the specific bathymetric features, and correcting for refraction, shoaling, frictional losses, island sheltering, and localized wind effects. Results are checked against measurements made during the period for which the hindcast is being made. The final product is a time history of nearshore directional wave spectra at 3-hour intervals for the 20-year period at each nearshore grid point (approx. 10 m depth). This massive computational effort consumed weeks of processor time on a supercomputer.

More details on the WIS wave hindcast at Bolsa Chica and a summary of results are provided in the comprehensive modeling report (Gravens, et al. in preparation).

Comprehensive Shoreline Response Modeling. The comprehensive shoreline response modeling task was similar to the preliminary shoreline modeling described above. The comprehensive modeling utilized the same modeling methodology as before, and much of the work performed in the preliminary task (e.g., bathymetry grids, shoreline position data, and model boundary conditions) did not have to be repeated for the comprehensive task. The key difference was that wave estimates from the WIS hindcast were used as input to the comprehensive modeling. These improved wave estimates provided more confidence in the numerical model results, and allowed model calibration to be

performed using actual hindcast waves for the calibration period rather than representative waves such as used in the preliminary modeling. Simulations of projected shoreline response, with and without sand management techniques, were performed as before. Completed results from the comprehensive shoreline response modeling task were not available at the writing of this paper, but they will be given in Gravens, et al. (in preparation).

#### BAY RESPONSE NUMERICAL MODELING

Purpose. The twofold purpose of the bay response modeling task was (1) to estimate the effects of the proposed ocean entrance alternatives on tidal circulation and constituent transport in the Bolsa Bay complex, existing and proposed wetlands, Huntington Harbour complex, and Anaheim Bay, and (2) to qualitatively assess impacts to water quality based on existing data and constituent transport estimates.

Scope. The scope of work for this task included the following:

- a. Evaluation of available numerical models and selection of the most appropriate model for application to the project.
- b. Gathering of existing and new field measurements necessary for the model study and water quality assessment.
- c. Calibration and verification of the tidal circulation numerical model to existing conditions.
- d. Application of the model to test sponsor-provided concepts for both navigable and non-navigable entrance alternatives.
- e. Assessment of water quality based on existing data and numerical transport simulations.

Methodology. The most suitable numerical model for application to Bolsa Chica and surrounding tidal regions needed to successfully simulate the flow characteristics of the channelized Anaheim Bay, Huntington Harbour, and Outer Bolsa Bay regions, and to satisfy the requirements of the water quality modeling effort. The selected model was a link/node model with the basic features of: inundation of low-lying terrain, treatment of hydraulic control structures such as culverts and tide gates, and utilization of actual bathymetry with spatially-varying bottom roughness. The link/node model divides the

channelized tidal region into small volumes (represented by nodes) joined together by links. Conservation of water mass is maintained throughout the tidal cycle simulation by transfer of water volume between adjoining nodes in the network.

Model calibration was achieved by reproducing the tidal elevations and water velocities measured in the existing Bolsa Bay region. It was found that the model was most sensitive to the geometry of the water channels and wetlands basins as the water capacity varied with the tidal elevation. This further reinforced the choice of the link/node model for application to this project because the proposed alternatives call for major expansion of the tidal wetlands acreage, and thus a significant change in tidal prism geometry. Complete details of model assumptions, calibration, and study results are given in Hales, et al. (in publication).

Tidal Circulation. The calibrated numerical model was used to examine a total of 12 variations of the two proposed entrance alternatives. These included the navigable entrance alternative with and without a navigable connector channel between Bolsa Bay and Huntington Harbour, the non-navigable entrance alternative, and four simulations where it was assumed that the non-navigable channel closed due to blockage by littoral material.

Water Quality Assessment. Existing data and information pertaining to water quality characteristics of Bolsa Bay were obtained from Federal, State, and Local agencies, and other organizations concerned with the water quality of the Bay. Supplemental field data were collected consisting of measurements of temperature, pH, conductivity, and dissolved oxygen. Sediment samples were gathered and analyzed to determine contamination in the existing wetlands. The link/node model was used to simulate transport of conservative tracer throughout the modeled tidal region, and these results provided estimates of water residence times for the existing Bolsa Bay configuration and for the various proposed alternatives. Finally, a qualitative assessment of potential impacts to water quality was made based on both the data analyses and the numerical simulations.

Study Elements. The bay response numerical modeling task specifically examined the following elements relative to potential impacts that may result from construction of a project at Bolsa Chica.

- a. The change of tidal flows and water quality in the Anaheim Bay.
- b. The water surface elevations in Huntington Harbour that would exist under either ocean entrance alternative.
- c. The effect of a navigable connector channel between Bolsa Chica and Huntington Harbour in terms of water flow in Huntington Harbour and Outer Bolsa Bay.
- d. The potential for scouring water flows in Outer Bolsa Bay due to closure of the non-navigable entrance.
- e. The amount of storm water runoff that would enter the wetlands if either alternative is constructed.
- f. The overall water residence time in the entire system when compared to existing conditions.
- g. The tidal flushing of Huntington Harbour under the various proposed alternatives.

#### ENTRANCE CHANNEL PHYSICAL MODELING

Purpose. The purpose of the physical modeling task was to examine wave penetration into the marina basin and the resulting harbor oscillations, to study qualitatively current circulation and sediment transport paths in the vicinity of the structures, and to make preliminary assessment of the entrance channel design configuration. In addition the physical model provided a tool for examining modifications to the incident waves caused by the protective structures so that surfing impacts could be assessed (see next section).

Scope. The scope of work for this task involved the following efforts:

- a. Design and construction of the physical model.
- b. Installation and calibration of the wave generator and pumps.
- c. Testing of the navigable entrance with and without a navigable connector to Huntington Harbor.
- d. Testing of the non-navigable entrance.
- e. Conducting sediment tracer tests and dye injection tests.
- f. Providing still and video footage of the physical model.



Model Testing. The physical model, as depicted on Figure 2, was constructed at a scale of 1-to-75. It reproduces 8,000 ft of the shoreline (110 ft in the model), and covers an area of approximately 2.8 sq mi (14,000 sq ft in the model). The model reproduces the Bolsa Chica bathymetry out to the 30 ft depth contour, and wave conditions are simulated using unidirectional irregular waves. The criteria used for allowable wave heights in the interior channels and basins were specified as 1 ft for the 1-year design event, and 1.5 ft for the 20-year design event. The first series of tests conducted resulted in design modifications to the navigable entrance system that met the wave penetration criteria.

Completed results from the physical modeling task were not available at the writing of this paper, but they are given in Bottin and Acuff (in prep.).

#### INLET STABILITY ANALYSIS

An analysis was performed to examine the stability of both the non-navigable and navigable ocean entrance channel alternatives being proposed for Bolsa Chica. Tidal prisms were calculated from numerical modeling simulations of tidal circulation within the proposed configurations for both alternatives. These values were used to apply the O'Brien (1931) criterion for equilibrium cross-sectional channel area.

The results indicated that the non-navigable entrance channel, as presently designed with training jetties terminating at the high water mark, appears to be larger than necessary to be maintained by the calculated tidal prism, and the entrance would be expected to decrease to a smaller cross-section. This would not represent a problem unless subsequent analysis of the tidal circulation in the bay indicates a reduced entrance throat area somehow degrades the circulation within the bay and decreases the water exchange between the bay and ocean. Greater concern was expressed about the ability of the channel to remain open under the action of littoral processes without the protection of a dual jetty system extending into the surf zone at least beyond the mean lower low water line. The possibility that the presently designed non-navigable entrance may close periodically or may require routine maintenance dredging should be a consideration in evaluation of this alternative.

The proposed navigable ocean entrance system, as designed, cannot be classed as a tidal inlet in equilibrium because the design is not based on



maintenance of the entrance by scouring water flows. The entrance instead is designed to prevent sediment from entering the inlet channel, thus making the entrance system a barrier to the major portion of longshore-moving sediment. Material that does enter the channel will be deposited, and periodic dredging may be required to maintain the entrance channel at its design dimensions.

#### IMPACTS TO SURFING

A qualitative assessment of potential impacts that an ocean entrance system at Bolsa Chica may have on local surfing activities was performed under contract. Existing surfing conditions were assessed by conducting interviews with local surfers and by examining wave results obtained from a Littoral Environment Observation Program conducted at Bolsa Chica. Based on knowledge of surfing and coastal processes, a method was developed for quantifying the incident wave climate in terms of desirable surfing qualities (Dally, in publication). Application of this method in assessing the proposed project impacts led to the following considerations:

- a. The primary impact to surfing is the potential loss of approximately 3200 ft of surf break due to the shadow zone of the detached breakwater. This zone would lengthen when the wave angle approach is very oblique. The impact zone will decrease with decrease in breakwater length.
- b. The loss of surf break is incurred only at times when surfable waves would otherwise be present, which was estimated to be less than 50% of the time.
- c. There is a possibility that wave reflection from the jetties may interact with non-surfable incident waves to form rideable waves.

Additional surfing assessment using the physical model of Bolsa Chica was performed, but results were not available at the writing of this paper. A complete description of the surfing analysis is included in the comprehensive modeling report (Gravens, et al. in preparation).

#### SUMMARY

Modern coastal engineering analysis tools have been used to assess and quantify, where possible, the impacts that two proposed ocean entrance alternatives would have if either were to be constructed at Bolsa Chica, California.

Technically, either proposed alternative is feasible in the engineering sense because it was shown that impacts to the adjacent shoreline and to the tidal wetlands could be mitigated. This would require sand management at a navigable entrance, and maintenance dredging at a non-navigable entrance. However, these technical findings must be added to many other issues under consideration before a final decision is made on the future of Bolsa Chica.

#### **RELATED COASTAL ZONE '89 PAPERS**

A special session on Bolsa Chica was held at Coastal Zone '89 that included seven papers spanning the historical, legal, developmental, federal, state, and regional issues. Scheduled authors were: J.F. Trout, D. Gorfain, and C. Fossum; D.A. Shelley; R.S. Joe; R.G. Fisher; J. McGrath; P. Green; and V. Leipzig.

The following related papers were scheduled for presentation at the Coastal Zone '89 Conference:

Gravens, M.B. "A New Ocean-Entrance System at Bolsa Chica, California -- Preconstruction Assessment of Potential Shoreline Impacts"

Kraus, N.C. "Beach Change Modeling and the Coastal Planning Process"

#### **ACKNOWLEDGEMENTS**

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#### **DISCLAIMER**

Engineering services were provided to SLC by WES under authority of Title III of the Intergovernmental Cooperation Act of 1968. As such, resultant study products are based on specific technical expertise only and should not be inferred to indicate support or non-support by the Corps of Engineers for the environmental or economic aspects of any subsequent project at Bolsa Chica.

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## SHORELINE CHANGE BEHIND TRANSMISSIVE DETACHED BREAKWATERS

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### ABSTRACT

This paper describes simulations of shoreline evolution behind detached breakwaters performed by using the shoreline change numerical model GENESIS. The model was recently enhanced to include wave transmission through breakwaters. Results of sensitivity tests are first presented, showing that GENESIS provides qualitatively reasonable predictions. Shoreline change at Holly Beach, Louisiana, site of six detached breakwaters of different transmissivities, is then successfully simulated in this first demonstration of the new capability.

### INTRODUCTION

Detached breakwaters provide an attractive and important shore protection alternative possessing different properties than groins and beach nourishment. Detached breakwaters may be used by themselves (either singly or in shore-parallel sections separated by gaps) or in combination with the traditional shore protection methods of groins and nourishment. Detached breakwaters reduce wave energy incident on the beach and impede the offshore transport of sand, neither of which properties are possessed by groins. Despite the advantages of detached breakwaters, they have been little utilized in the United States as compared to other countries, in particular, Japan, Spain, and Israel.

The planning and design of a detached breakwater system requires consideration of many factors, including structure length, distance offshore, crest height, composition of the core of the breakwater, and gap width in the case of segmented breakwaters. These parameters must then be related to the average and extreme wave heights, wave direction, profile shape, and tidal

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variation in order to estimate the shore protection capabilities of the breakwater system. Perlin (1979) investigated geometric parameters determining the influence of a breakwater on the shoreline by using a numerical simulation model of a single shore-parallel structure. Kraus (1983) obtained good agreement in a comparison of breaking wave height and direction and shoreline change behind a detached breakwater calculated with a numerical model and measured in a physical model. Kraus and Harikai (1983), Kraus, Hanson, and Harikai (1984), and Hanson and Kraus (1986a) modeled waves and shoreline change behind large breakwaters in the field, and Hanson (1987, 1989) modeled shoreline change measured behind three detached breakwaters at Lakeview Park, Lorain, Ohio.

All of the aforementioned numerical modeling studies reproduced correct trends in shoreline evolution behind detached breakwaters. However, an important process absent in these works was wave transmission through the breakwaters. Wave transmission is a decisive factor in most practical applications, since it is economical and often advantageous from the perspective of beach change control to build low and/or porous structures which allow a portion of the incident wave energy to penetrate directly behind them. The shoreline change model GENESIS (Hanson 1987, 1989; Hanson and Kraus 1989) has recently been enhanced to include wave transmission at detached breakwaters, and the purpose of the present paper is to demonstrate this new capability. Technical details will be given in Hanson and Kraus (in prep).

#### DETACHED BREAKWATER PROCESSES

The most obvious shore protection property of detached breakwaters is the wave sheltering afforded to the beach (Fig. 1). The wave height and longshore current speed are reduced behind these structures, and sand carried by the longshore current is deposited in the calm "shadow zone," resulting in seaward progression of the shoreline. If a detached breakwater is placed too far offshore, its sheltering effect will be inoperative, whereas if it is placed too close to the shore, the beach will prograde excessively, forming a tombolo.

Typically, the most desired structure placement is such that the resultant cusped-shaped equilibrium beach form, called a salient (Dally and Pope 1986),

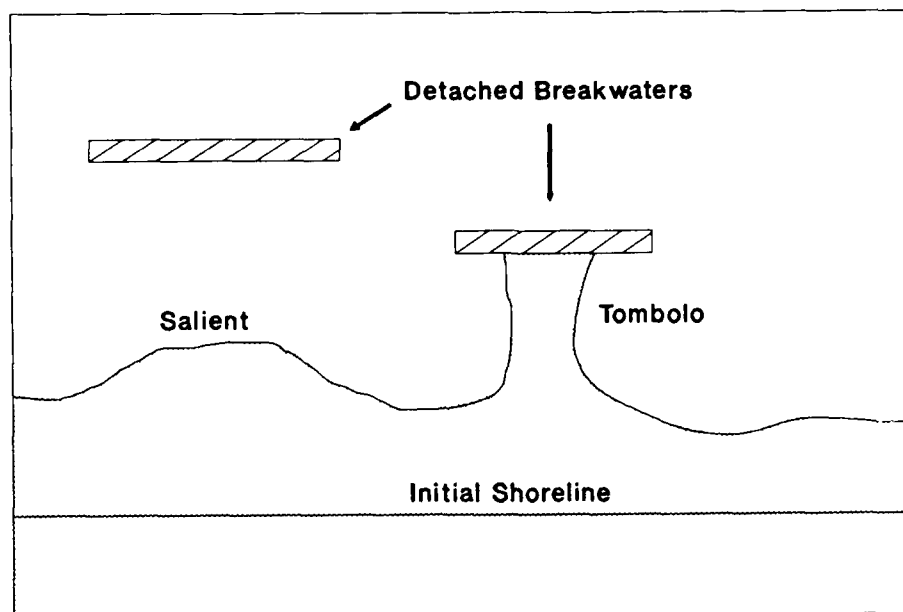


Fig. 1. Schematic of shoreline change behind detached breakwaters

does not reach the breakwater. This allows sand to be transported alongshore between the structure and the beach and to reach downdrift beaches, yet the shore remains protected at the site. In some situations, headlands produced by tombolo development may be the design aim. Pope and Dean (1986) provide empirical guidance based on the functioning of detached breakwaters in the United States, permitting an estimate to be made as to whether a tombolo or salient will form, or if no shoreline change will occur (see also, Suh and Dalrymple, 1987). It is expected that (1) placement closer to the shore will promote tombolo development, (2) longer breakwaters will promote tombolo development, and (3) greater wave transmission will inhibit tombolo development.

Essentially all detached breakwaters built for shore protection transmit wave energy by (1) wave passage over the structure, called overtopping, during times of relatively high water level and/or high waves, and (2) wave passage through the structure (depending on the composition of the breakwater). Here, these two processes will be collectively referred to as "wave transmission."

As a structure settles, wave transmission will increase. Wave transmission improves water circulation, limits seaward growth of the salient, and reduces wave forces on the structure, thereby increasing its longevity.

A wave transmission coefficient  $K_T$  can be defined as the ratio of the height of the incident waves (prior to reflection) to the height of the wave immediately behind the breakwater. The value of  $K_T$  thus ranges between 0 and 1, with the value 0 indicating an infinitely high impermeable breakwater and the value 1 indicating complete transmission (no breakwater). At present, the value of  $K_T$  is specified empirically for use in the model.

#### WAVE DIFFRACTION

Waves incident to a detached breakwater diffract at the tips of the structure, and wave energy is transferred behind it (Fig. 2). The nearshore area adjacent to the structure and directly reached by waves is called the illuminated region, and the area sheltered from direct wave incidence and reached solely by waves radiating from the tips of the structure is called the shadow region. The change in wave direction at each tip and the decreasing wave height with distance alongshore behind the structure both produce a longshore current directed into the shadow region. Sand transported alongshore by the current is deposited in the calmer wave shadow region behind the breakwater. In Fig. 2, the reduced wave height is represented by decreasing values of the diffraction coefficient  $K_D$ .

The longshore sand transport rate is normally expressed in terms of wave conditions at breaking. To reproduce shoreline change behind detached breakwaters, therefore, both wave diffraction and transmission must be accurately represented in the breaking wave calculation. A pragmatic procedure incorporated in GENESIS to calculate breaking wave height and direction under combined wave diffraction, refraction, and shoaling was developed by Kraus (1983, 1984). The capability to calculate wave transmission together with these transformations was recently added to the model (Hanson and Kraus, in prep).

Diffraction and transmission are interdependent. As waves propagate over and through a detached breakwater and into the shadow region, the difference in wave height between the illuminated and shadow regions will decrease, and thus also the effect of diffraction. As a consequence, less sand will be

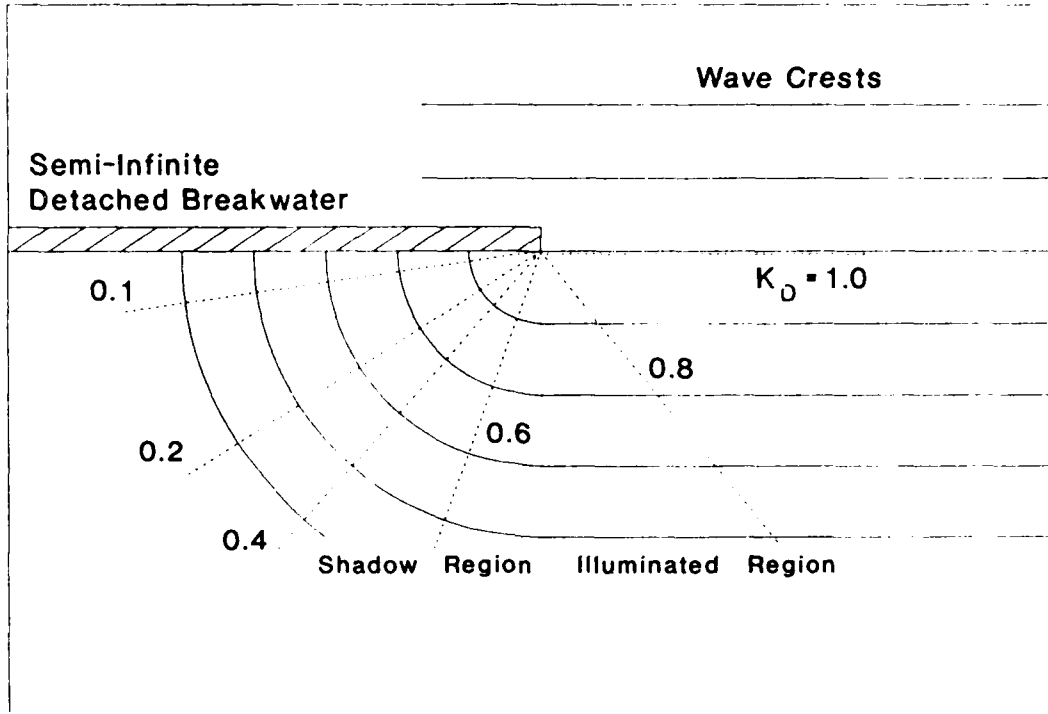


Fig. 2. Diffraction behind a semi-infinite breakwater (schematic)

transported alongshore and into the area behind the structure. In addition, the transmitted waves themselves will tend to transport sand out of the area behind the detached breakwater. Thus, these two mechanisms (reduced transport from diffracted waves and direct transport of sand by transmitted waves) act together to suppress growth of the salient behind the breakwater.

#### NUMERICAL MODEL

GENESIS is a finite-difference numerical model developed to simulate shoreline change produced by wave action (Hanson 1987, 1989; Hanson and Kraus 1989). The model calculates shoreline change occurring over a period of months to years and with a length scale varying from one to tens of kilometers (Kraus 1983, 1989). GENESIS simulates shoreline change for a wide variety of user-specified beach and coastal structure configurations (for example, Hanson, Gravens, and Kraus 1988).

Version 2.0 of GENESIS, presently under testing, allows representation of wave transmission at detached breakwaters. Each breakwater can be assigned a



separate transmission coefficient. Detached breakwaters with variable wave transmission alongshore (due, for example, to structure deterioration and settling) can be modeled by several smaller contiguous sections having different transmission coefficients. Gaps between breakwaters as well as detached breakwaters which allow wave transmission ( $K_T > 0$ ) are called "energy windows" as they represent areas in the offshore through which wave energy can directly penetrate.

#### Treatment of Transmission and Diffraction

Wave transmission is assumed to possess the following properties, which will be used to examine model predictions in the examples below:

- a. As the transmission coefficient approaches zero, calculated wave diffraction should equal that given by standard diffraction theory for an impermeable infinitely high breakwater.
- b. If two adjacent energy windows have the same transmission coefficient, (for example, two detached breakwaters with the same  $K_T$  or one breakwater with  $K_T = 1$  situated next to a gap), no diffraction should occur.
- c. On the boundary between two energy windows with different transmission coefficients, wave energy should be conveyed from the window with higher waves to the window with smaller waves. The amount of wave energy transferred should be proportional to the ratio between the two transmission coefficients.

Fig. 3 shows the calculated distribution of breaking wave height behind a semi-infinite detached breakwater as a function of transmission. The breakwater is located 250 m from the shoreline. The wave period is  $T = 6$  sec, and the incident wave crests are parallel to the straight shoreline. The breaking wave height  $H$  alongshore is normalized by the wave height at the tip of the breakwater,  $H_{tp}$ . The curves labeled by a denote breaking waves incident from the lateral side of the structure, and b denotes waves transmitted through the structure. The total wave height is obtained as the sum of these two wave systems, denoted by c. (Calculated breaking wave angles and long-shore sand transport rates are discussed in Hanson and Kraus, in prep.)

For  $K_T = 0.0$ , only diffraction occurs, and no waves pass through or over the structure. The curves a and c are therefore identical.

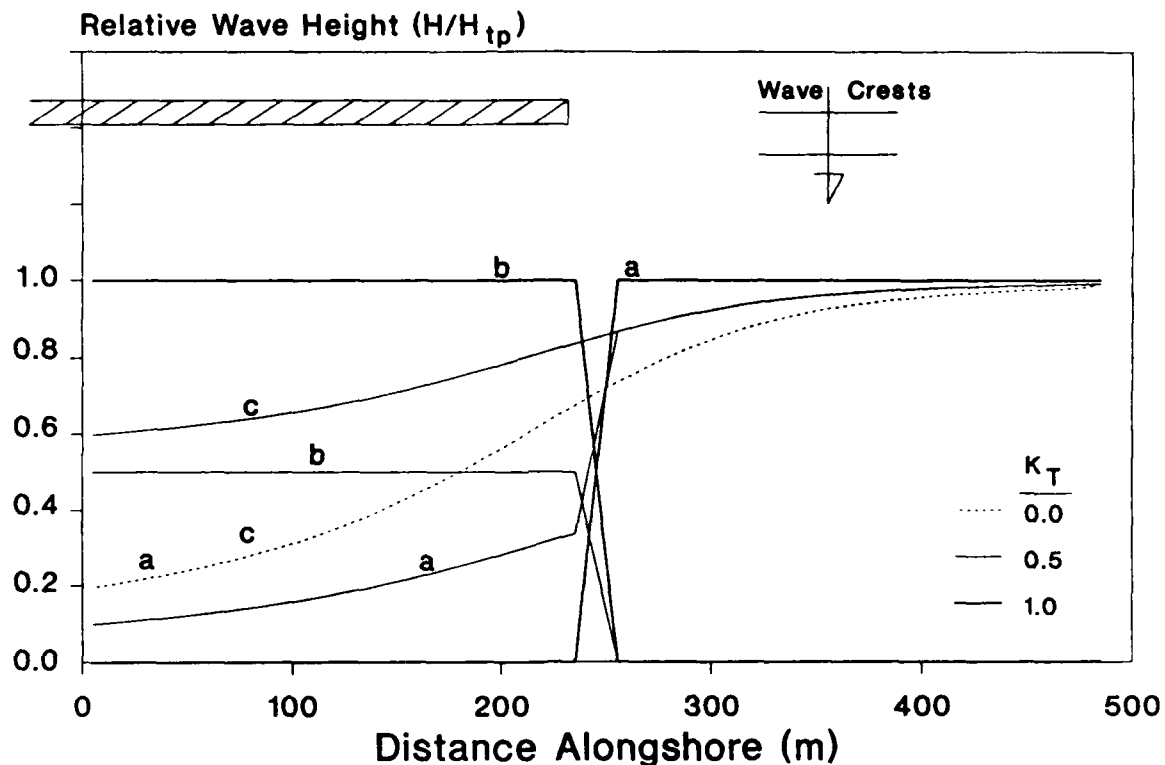


Fig. 3. Distribution of relative breaking wave height,  $H/H_{tp}$

For  $K_T = 0.5$ , the relative wave height in the shadow region is 0.5. Since the wave height in the shadow region is now greater as compared to the case of no transmission, wave diffraction in the illuminated region must weaken. As shown by curve *a*, the wave height to the right of the structure is half-way between the curve for pure diffraction and the curve depicting no diffraction ( $H/H_{tp} = 1.0$ ). Since diffraction acts to transfer energy from areas of higher waves to lower waves, waves transmitted through the breakwater will not diffract into the illuminated region, and the transmitted wave height *b* drops sharply from 0.5 to zero. The alongshore distribution of the combined wave height *c* meets line *a* in the illuminated region, half-way between the diffraction curve and the curve  $H/H_{tp} = 1.0$ .

For  $K_T = 1.0$ , waves incident to the breakwater pass undiminished. Diffraction does not occur, and wave heights in the shadow and illuminated regions are equal. The relative wave height on either side of the separation

line is unity up to the calculation cell adjacent to the grid cell of the diffracting tip. At that cell, the relative wave height is 0.5, dropping to 0.0 in the cell past the tip. Therefore, the total relative wave height is 1.0 along the entire shoreline. For clarity, the corresponding line  $C$  is not shown in Fig. 3.

#### Influence of $K_T$

Fig. 4 shows the case of a 200-m long detached breakwater located 250 m offshore. Conditions are:  $H = 1.5$  m,  $T = 6$  sec, wave crests normal to the initially straight shoreline, and simulation time of 180 hr. As expected, the seaward extent of the salient decreases as wave transmission increases. Also, the salient broadens slightly with increased transmission, and the eroded areas on either side of the salient fill in. A simulation performed with  $K_T = 1.0$  produced no shoreline change and is not shown in Fig. 4.

#### Breakwater Segments with Different Transmission

Fig. 5 shows a three-breakwater system with asymmetrical wave transmission properties, the greatest transmission assigned to the right-hand breakwater and least to the left-hand breakwater. The calculation is significantly more complex than in the previous examples, because a point on the beach is open to seven wave energy windows (four gaps and three transmitting breakwaters). The curves in Fig. 5 display results for four cases with  $H = 1.5$  m and  $T = 6$  sec distinguished by wave direction (0, +10, -10, and  $\pm 10$  deg); the direction was constant for 120 hr for the first three cases, and in the fourth case the angle switched from +10 to -10 deg at the midpoint of the 120-hr simulation.

The most obvious feature of Fig. 5 is the significant size difference of the salients. The size and location of the largest salient is relatively independent of wave direction, confirming conclusions of Hanson and Kraus (1986a), who found that in diffraction-dominant situations, the response of the shoreline behind breakwaters is most sensitive to incident wave height and not wave direction (since almost identical semicircular diffraction wave patterns are formed for any reasonable direction of the incident waves). In contrast, the calculated shorelines in the lee of the middle and right breakwaters show substantial differences, similar to the open-coast situation in which oblique wave incidence controls the direction of sand transport.

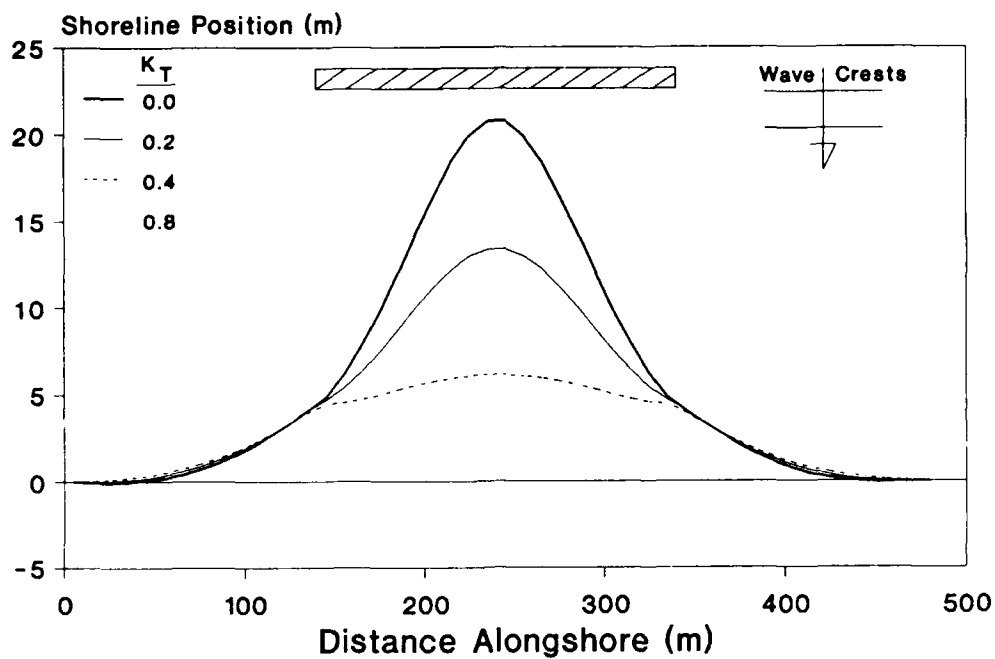


Fig. 4. Shoreline change as a function of transmission

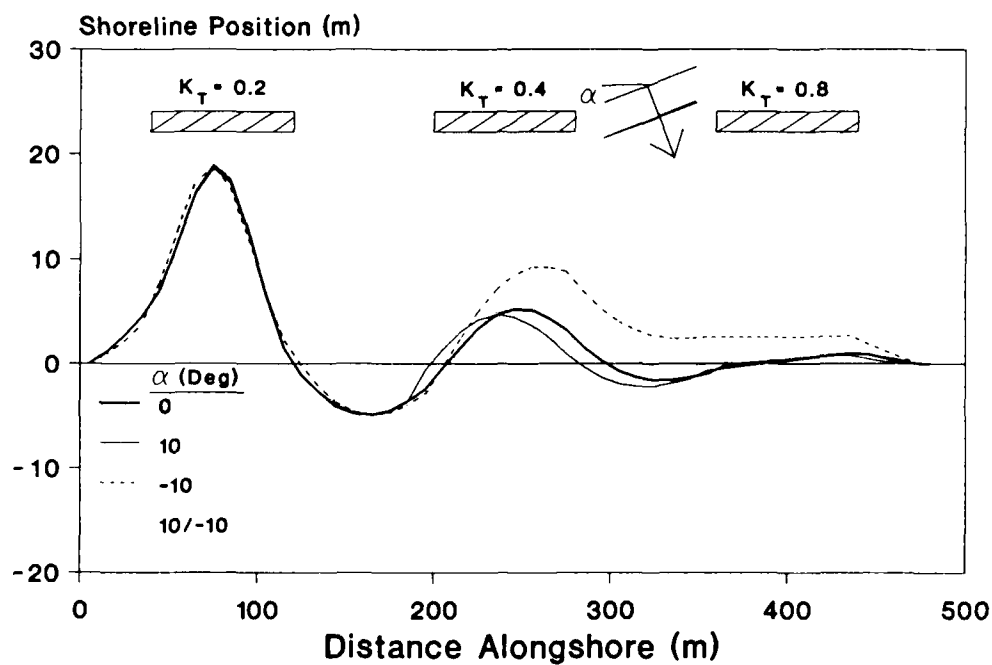


Fig. 5. Shoreline change behind three detached breakwaters of different wave transmission properties

The beach planform behind the middle breakwater is asymmetric, being sheltered and sand-deprived on the left side and more open to wave energy on the right side. Shoreline recession is more severe between the left and middle breakwaters than between the middle and right breakwaters because diffraction is stronger. For the -10-deg case, where sand tended to move from right to left in the figure, substantial accretion resulted from blockage (groin effect) by the middle salient. (In all examples, a "pinned beach" condition was applied on the lateral boundaries, allowing sand to freely enter and leave the modeled area.)

#### Variable Transmission Breakwater

Detached breakwaters with variable transmission properties alongshore can be represented by contiguous sections having different transmission coefficients. The configuration shown in Fig. 6 illustrates calculated shoreline change produced by two semi-infinite detached breakwater segments with transmission coefficient  $K_{T1}$  connected by a segment with transmission coefficient  $K_{T2}$ . This situation mimics the central area of a very long breakwater which might have experienced damage, altering its transmission properties. The breakwater was located 100 m from the initially straight shoreline, and the offshore wave conditions were  $H = 1.0$  m and  $T = 6.0$  sec, with the wave crests arriving parallel to the breakwaters. The simulation time was 120 hr.

Figure 6 shows that sand was transported away from the beach behind breakwater sections with greater transmission and deposited behind areas protected by breakwater section(s) with less transmission. The shoreline change was delicately controlled by the opposing sand transporting mechanisms of diffraction and transmission, and became more sinuous with increasing difference in transmission coefficients. A double salient tended to form, which is a possible shoreline response behind detached breakwaters and depends on the ratio of distance between the breakwater and initial shoreline and the ratio of structure length to wavelength. Perlin (1979) obtained double salients in simulations neglecting transmission, and a subaqueous double salient was generated by a detached breakwater in the physical model experiment performed by Mimura et al. (1983).

The example calculations demonstrated that GENESIS produces reasonable trends of shoreline change for general combinations of wave transmission,

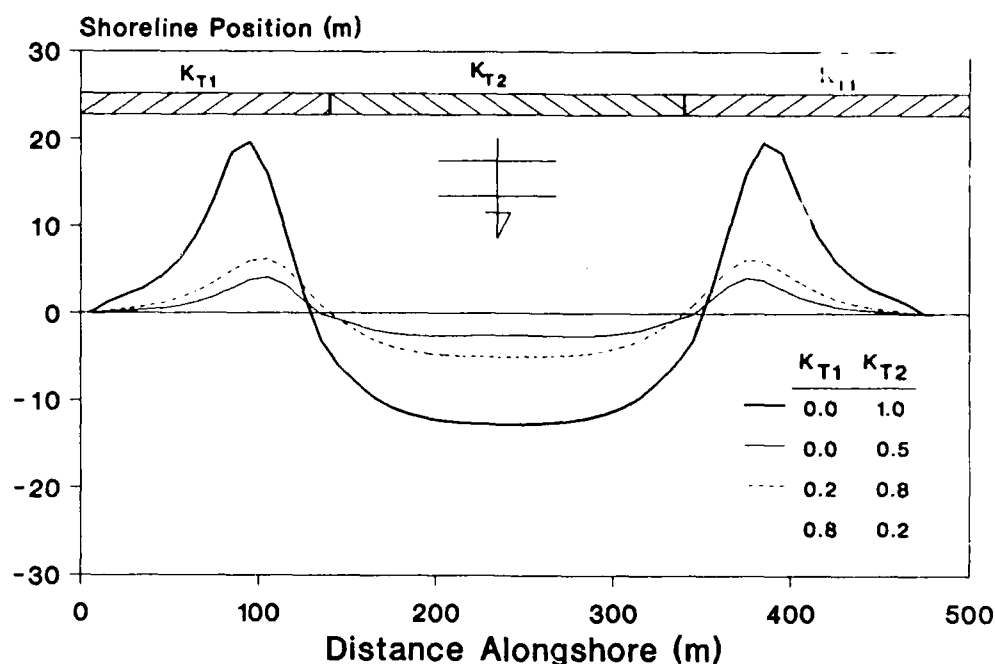


Fig. 6. Shoreline evolution behind a detached breakwater with variable wave transmission

giving strong support to the newly developed wave calculation procedure. As an empirical test of the enhanced model, a simulation was made of the shoreline evolution which occurred after installation of six detached breakwaters at Holly Beach, Louisiana.

#### APPLICATION TO HOLLY BEACH, LOUISIANA

##### Site Description

Holly Beach is located near the Texas - Louisiana border on the chenier plain along an east-west oriented coast (Fig. 7). This coast is undergoing chronic erosion resulting from limited sand supply, relative sea level rise caused in part by regional subsidence, and seasonal impacts from frequent extratropical cyclones (cold fronts) occurring in winter and hurricanes developing in the late summer and early fall. The wave energy is typically low, with average breaking wave height of 50 cm and a period of 5 sec (Nakashima et al. 1987). The tidal environment is microtidal; the diurnal tides have a mean range of 60 cm and a spring tide range of 74 cm. Annual wind data indicate that locally generated wind waves dominate from the south

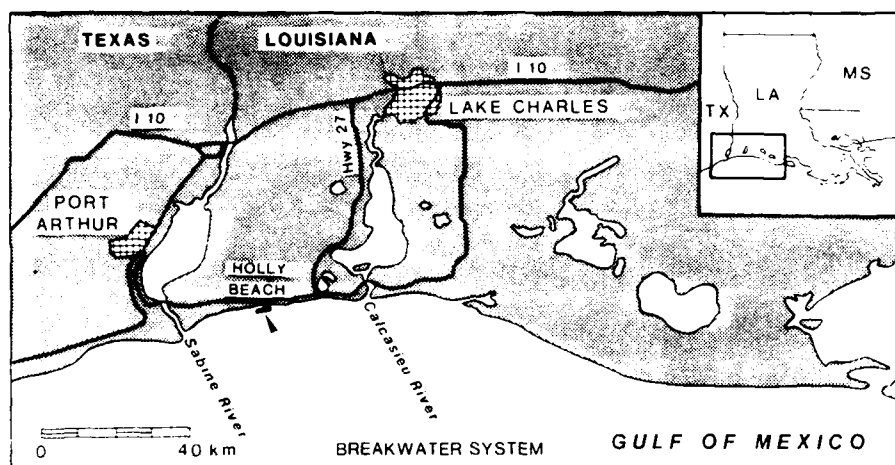


Fig. 7. Site map for Holly Beach, Louisiana, breakwater project

and southeast quadrants 18 and 22 percent of the time, respectively. The nearshore is morphodynamically dissipative with slopes ranging from 0.03 to 0.05 (Nakashima 1989). The gentle slope limits breaking wave angles. Small wave angles and moderate wave heights result in a low longshore sand transport rate, which has been estimated by the U.S. Army Corps of Engineers (1971) at 47,000 to 76,000 m<sup>3</sup>/year to the west. Sediments composing the beaches in the Holly Beach area are fine sands with a mean of 0.20 mm and well sorted at 0.72 mm (standard deviation).

Since 1969, various measures have been taken to protect State Highway 82, which parallels the coast and serves as the hurricane evacuation route for communities to the west of Holly Beach. In 1970, a 5-km long revetment was constructed to protect the highway, and in 1973 the highway was damaged by Hurricane Delia. Various types of restoration materials, including concrete blocks and small quantities of beach fill, have been placed along the shore in attempts to protect the highway. The revetment and highway have been damaged

periodically by hurricanes in subsequent years and portions of the highway have been rebuilt landward. The most recent maintenance took place during early 1988, when concrete-filled bags and boulders were used to stabilize the revetment to the west of the breakwater system constructed in January, 1986. The eastern end of the revetment was reinforced in a similar manner with a combination of mats of concrete blocks and boulders. This additional attempt at stabilization was necessary because the highway presently fronts lower elevation back marshes and cannot be relocated landward without considerable coast.

The breakwater system consisting of six segments represents a significant departure from a revetment as a shore protection measure. Although originally conceived as a T-groin system, this project was modified to a series of segmented detached breakwaters consisting of various quantities and arrangements of timber piles, used tires, and riprap, which provided different amounts of wave transmission (Fig. 8). By allowing some degree of wave transmission, the longshore movement of sand would not be completely interrupted at the breakwaters, and downdrift erosion of the natural shoreline would be reduced. The breakwaters were constructed as a prototype test of a low-cost shore protection method with the multiple objectives of preserving and possibly widening the narrow beach, protecting the revetment, and reducing wave overtopping and flooding of the highway. The concern for downdrift beaches in addition to the critical eroding area at the breakwater site is an example of comprehensive shore protection project planning as discussed by Kraus (1989).

The Louisiana Geological Survey (LGS) and the Coastal Engineering Research Center (CERC), U.S. Army Engineer Waterway Experiment Station, initiated a monitoring program consisting of periodic vertical aerial photography, quarterly beach profile surveys, and visual observation of local waves and nearshore circulation. This included pre-project aerial photography and surveys to establish preproject conditions. Nakashima et al. (1987) qualitatively evaluated the breakwaters to have approximately increasing transmissivity from west to east, with the rock riprap breakwater at the west end allowing the least transmission and the breakwater at the east end (tires





Fig. 8. Aerial photograph of the Holly Beach breakwaters (March 1986)

mounted on one row of timber piles) having the greatest transmission. During typical wave conditions, the riprap unit to the west showed no transmission, but, because of its low crest height, it was observed to be overtopped during storms.

The west (riprap) breakwater was placed 78 m offshore and the other five breakwater approximately 62 m offshore. The breakwaters are nominally 50 m long with a gap width of approximately 90 m and have effective crest heights ranging from 1.2 to 1.8 m above MSL. Within a few months after construction, large salients had formed behind the three western structures of lower wave transmission. These shoreline forms showed considerable movement and deformation with passage of Hurricane Bonnie on 26 June 1986. All salients remained intact but shifted 50 m to the east and decreased in their seaward extent by 30 to 70 percent. The eastern-most breakwater constructed of a single row of timbers incurred major damage during Hurricane Bonnie, but regeneration of a small salient subsequent to the storm nevertheless occurred.

## Numerical Simulation

Input data and model preparation. Input data requirements for shoreline modeling are discussed by Kraus (1989). A straight longshore baseline was drawn running down State Highway 82, and the locations of the breakwaters, revetment, and shoreline positions as determined from aerial photographs and profile surveys were referenced to the baseline. The revetment prevents the beach from retreating landward and was represented as a seawall constraint (Hanson and Kraus 1985, 1986b). Longshore model grid spacing of 4.6 m (15 ft) was used to provide approximately 10 calculation points behind each detached breakwater. Measured shoreline positions from the 24 irregularly spaced profile survey lines were transformed to the grid by using a nonlinear interpolation technique. The total model reach was 1,066 m (3,500 ft). A pinned boundary condition (determined from aerial photographs showing locations of minimal movement of shoreline position) was applied on both ends of the model grid to allow sand to be freely transported into and out of the calculation domain (Hanson and Kraus 1989).

Wave height and period at 1-hr intervals were obtained from a resistance wave gage on an oil platform located 140 km to the south of the study area in a water depth of 12 m. Wave direction was inferred from 1-hr records of wind direction measured on the same platform. These data cover the period of Hurricane Bonnie and were used to provide input to GENESIS to simulate beach change between the profile surveys of 1/23/86 and 7/29/86. The gage wave heights are believed to be an overestimation of the waves that arrived to the site because Bonnie made landfall close to the gage; also, in the modeling, the waves were assumed to propagate without dissipation. Wave heights at the gage were therefore halved and the data extensively censored to eliminate apparent spurious extremes to give a mean wave height of 0.53 m and  $T = 5$  sec at the gage. The offshore bathymetric contours were assumed to be straight and parallel, a reasonable approximation for this coast.

Preliminary calibration. Calibration refers to adjustment of model parameters to reproduce shoreline change that occurred between two surveys. At the time of writing, model calibration is in progress, so that the result shown should be considered as preliminary. In calibrations performed without wave transmission, only two empirical coefficients are adjusted, which deter-

mine the longshore sand transport rate and resultant shoreline evolution. These were set to  $K_1 = 0.5$  and  $K_2 = 0.1$  after initial testing (See, for example, Kraus (1983), or Hanson (1987, 1989) for further discussion of these parameters.)

In the present case, a wave transmission coefficient was assigned to each of the six breakwaters as part of the calibration procedure. Initial trial estimates of  $K_T$  were made on the basis of information provided by Nakashima et al. (1987). In order from east to west, the initial assigned values were: 0.9, 0.3, 0.5, 0.3, 0.7, 0.1. During the calibration, these were modified to be: 0.4, 0.8, 0.2, 0.1, 0.0, 0.0. Modification of the original values was expected since they were inferred from visual observation of wave and dye movement, whereas the transmission coefficients in the model pertain to wave heights and directions (wave energy fluxes) associated with combined wave diffraction and transmission.

The result of the preliminary calibration is shown in Fig. 9. The measured shoreline of 1/23/86 was used as the initial shoreline in the model. (It is interesting to note that this survey was made shortly after project construction and that salients had already begun to form.) Overall, the calculated shoreline position agrees well with the measured position of 7/29/87, demonstrating the importance of incorporating wave transmission at breakwaters in shoreline simulations. The locations of the tips of salients and their widths are well reproduced, whereas the calculated indentations in the shoreline between the salients are somewhat less pronounced than were produced in nature.

Epilogue on field monitoring and model predictions. The quarterly profile survey data of 12/88 indicate that the greatest beach development is in the form of an intertidal tombolo at the breakwater having the least wave transmission (western unit). The salient at the breakwater next to this one has undergone persistent accumulation, with the apex of the salient terminating 10 m from the breakwater. However, all but 20 m of this salient is subaqueous. The salient at the third breakwater from the west differs from the others in that sediment has been restricted to a beach without formation of a small-scale subaqueous tombolo. The quantity of sediments deposited to the lee of

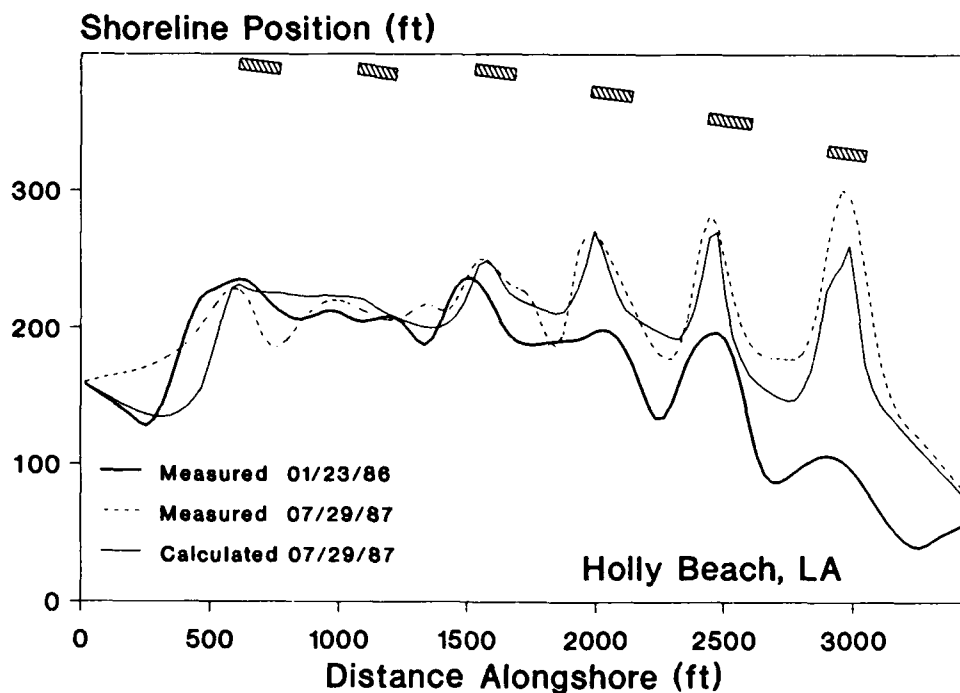


Fig. 9. Preliminary results of model calibration

the three eastern breakwaters has remained about the same as that measured in the 1987 surveys, with only small salients apparent.

Predictions of shoreline change with the calibrated model do not depart from the 12/88 survey that showed a persistent elongation of the salients for the three western breakwaters and a reduced or negligible amount of sediment accumulation behind the three eastern structures. As a further exercise, on the assumption of continued deterioration of the breakwaters, transmission coefficients were increased by 0.1, except for the severely damaged easternmost breakwater, which was assigned  $K_T = 0.9$ . After three years of simulation, the shoreline at the three eastern breakwaters retreated to the revetment, whereas the salients at the three western breakwaters persisted, but with smaller areas, closely reproducing qualitative features of the 12/88 survey.

## SUMMARY AND CONCLUDING DISCUSSION

The use of detached breakwaters is expected to increase as a shore protection measure because of their desirable characteristics of (1) reducing wave energy, (2) increasing the beach width, even under conditions favoring offshore sand transport, (3) allowing longshore movement of sand, and (4) maintaining water circulation. A shore protection project involving detached breakwaters necessarily requires consideration of a large number of parameters. These factors must be identified and their relationship to the functioning of a detached breakwater design understood to properly examine breakwaters as one of the alternatives in the planning process for shore protection. Numerical models developed to simulate shoreline change have proven to be a powerful tool for planning in the coastal zone, in particular, for evaluating the functioning of alternative designs of protective coastal structures and beach nourishment (Kraus 1989). Inclusion of wave transmission further enhances usefulness of these models.

Detached breakwaters do have disadvantages which must be considered in the evaluation of alternative shore protection plans. These are mainly (1) the relatively high construction cost, (2) loss of beach area to direct wave action, as required for surfing (although the shadow zone does provide a calm bathing area for weak swimmers, such as children), (3) to some, the unaesthetic interruption of the ocean view, and (4) the complexity of determining the appropriate design to obtain a properly functioning breakwater system. To reduce construction costs and to control shoreline impacts, detached breakwaters can be built with low crest heights and permeable cores. Other advantages accrue from use of low and porous breakwaters, including reduction of wave force on the structure (implying lower maintenance cost) and improved water circulation.

In order to estimate the impacts of detached breakwaters, wave transmission must be taken into account in most situations of practical interest. This paper has demonstrated the newly developed capability to model the effects of wave transmission together with the other major factors necessary to arrive at comprehensive and quantitative estimates of the functioning of detached breakwaters for shore protection. Tests of the transmission algorithm gave intuitively reasonable results, and the shoreline change model

GENESIS produced excellent agreement in preliminary calibration runs to simulate measured shoreline change behind six prototype detached breakwaters of different transmissivities.

Future modeling plans call for further tests of GENESIS. The model will then be used to develop general design criteria which will allow coastal managers and engineers to make initial estimates of breakwater placement as a function of environmental and design parameters.

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A NEW OCEAN-ENTRANCE SYSTEM AT BOLSA CHICA BAY, CALIFORNIA:  
PRECONSTRUCTION ASSESSMENT OF POTENTIAL SHORELINE IMPACTS

Mark B. Gravens<sup>1</sup>

**ABSTRACT**

This paper describes the use of the shoreline change numerical model GENESIS in the assessment of potential shoreline impacts resulting from the construction of a structured inlet entrance system at Bolsa Chica, California. The methodology of shoreline change modeling, including the preliminary steps of data collection, analysis, and preparation for input to the shoreline change model, is discussed, as well as interpretation of models results. This paper illustrates the utility of shoreline change models in the assessment of shore protection alternatives and the modification of longshore sediment transport processes.

**INTRODUCTION**

The shoreline change study discussed herein was conducted by the U.S. Army Engineer Waterways Experiment Station (WES), Coastal Engineering Research Center (CERC), for the California State Lands Commission (SLC). This study was performed as one task of a multi-tasked engineering investigation which examined the impacts that a proposed ocean entrance system at Bolsa Chica, California, would have on the adjacent shorelines and tidal wetlands. An overview of the suite of studies conducted at WES is presented in a companion paper (Hughes 1989).

The purpose of the shoreline modeling effort was to quantitatively assess the potential long-term impacts of the proposed ocean entrance system at Bolsa Chica on adjacent shorelines and to investigate the potential for mitigation of any adverse effects induced by the entrance. Three major components were involved in the shoreline modeling effort: (1) preliminary shoreline response study, (2) comprehensive wave hindcast, and (3) comprehensive shoreline response study. In the preliminary shoreline response study, available shoreline and wave data were collected, analyzed, and input to the shoreline change numerical model to predict the response of adjacent

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shorelines to the introduction of a littoral barrier in the local littoral cell. The comprehensive wave hindcast effort involved a 20-year hindcast of wave conditions for the Bolsa Chica region. The hindcast provided estimates of locally-generated wind sea and Northern Pacific swell wave conditions at 3-hour intervals for the 20-year hindcast period 1956 to 1975. A supporting effort was an 18-month hindcast of Southern Pacific swell wave conditions, which provided an estimate of the importance of this component of the local wave climate.

After the wave hindcasts were complete, a comprehensive shoreline response modeling effort was initiated in which the hindcast wave estimates were used as input to the shoreline change model. A description of the shoreline response tasks, details of the wave hindcasts, and the overall study results is presented in Gravens 1988, and Gravens (1990).

#### STUDY AREA

Bolsa Chica is located in southern California in an unincorporated area of Orange County about 9 miles south of Long Beach and is surrounded by the City of Huntington Beach (Figure 1). The site of the proposed entrance system is located approximately 3 miles south of Anaheim Bay and 7 miles north of the

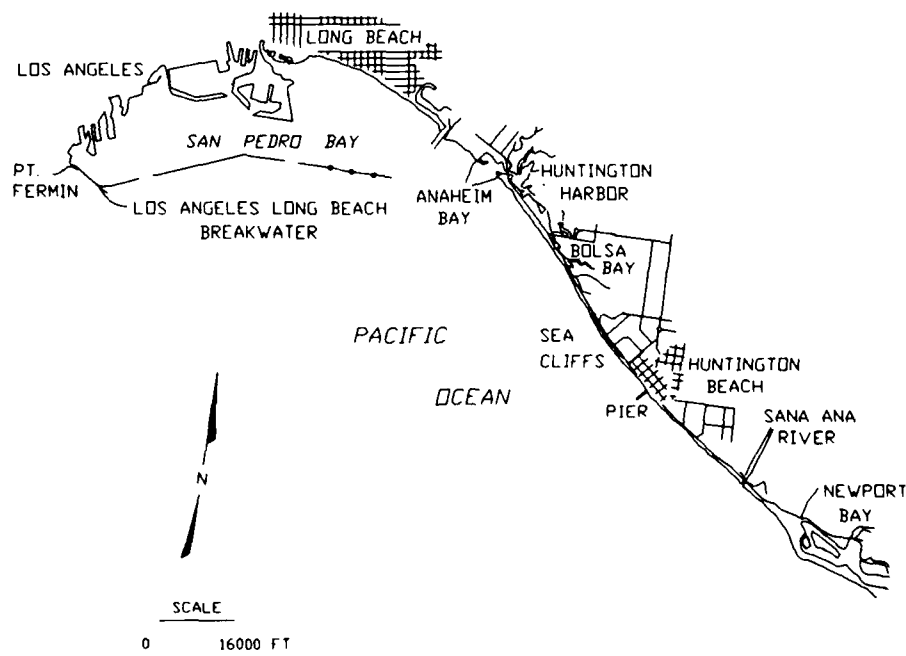


Figure 1. Bolsa Chica study area

mouth of the Santa Ana River. This region is referred to as the Newport Littoral Cell (Inman 1976, Hales 1984, U.S. Army Corps of Engineers 1978, 1987). The northern limit of the littoral cell is at Anaheim Bay, which acts as a complete barrier to the movement of littoral material alongshore. The littoral cell terminates at the Newport Submarine Canyon offshore of Newport Beach. A littoral cell is defined as a coastal segment that contains a complete sedimentation cycle including sources, transport paths, and sinks. The Newport Littoral Cell satisfies these requirements; the sources are the feeder beach located immediately east of Anaheim Bay (Surfside-Sunset Beach), and the infrequent transport of sediment to the beach by the Santa Ana River to the south of Huntington Beach. The transport path is the surf zone energized by breaking waves, and the ultimate sinks to the southeast are the Newport Submarine Canyon and the steeper nearshore bathymetry of the Newport region. Other potential depositories of sand are beaches along the cell and the beach profile in areas where extraction of oil has caused local subsidence (Woodward-Clyde Consultants 1984, 1986). Beaches between Anaheim Bay and Santa Ana River have accreted an average of 4.4 ft/year for the period 1934 to 1983.

The approximately 10-mile-long shoreline reach from Anaheim Bay to the Santa Ana River was modeled using a numerical model of shoreline change. Coastal structures and features of importance within the model reach include the east Anaheim Bay jetty, the sea cliffs at Huntington Beach, the Huntington Beach pier, and the north jetty at the mouth of the Santa Ana River. Each of these features influences the evolution of adjacent shorelines and was represented in the shoreline change model. The sea cliffs at Huntington Beach (a remnant of a historical headland formerly extending seaward of the present shoreline) serves to pin the shoreline between the cliffs and Anaheim Bay to the northwest and the Santa River to the southeast. The Huntington Pier and the east Anaheim Bay jetty modify the local breaking wave pattern and produce a local shoreline signature unique to these structures. Prior to initiating the numerical shoreline change simulations, considerable analysis of existing physical data were performed as described in the next section.

## INPUT DATA

Kraus (1989) provides a summary of data requirements for shoreline change modeling. Here, major data inputs for the present project will be described.

### Shoreline Position Data

Maps containing historical shoreline position data (from surveys) were obtained from the U.S Army Corps of Engineers, Los Angeles District. These data, summarized in Table 1, were digitized at approximate 100-ft intervals with respect to a baseline oriented along the general trend of the shoreline (on a northwest/southeast line), and cubic spline interpolation was used to produce shoreline positions with an exact alongshore spacing of 100 ft to allow direct comparison and further manipulation. An analysis of the shoreline position data was performed to summarize spatial and temporal variabilities. Mean, standard deviation, and average absolute shoreline change were calculated for four regions within the study area. The shoreline segments are: Segment 1, Santa Ana River to Anaheim Bay (modeled reach); Segment 2, Santa Ana River to Huntington Pier; Segment 3, Huntington Pier to Anaheim Bay; Segment 4, near proposed ocean entrance site (a 2.7-mile-long reach centered about the proposed entrance system).

Table 1  
Summary of Shoreline Position Data Sets

<u>Date of Survey</u>	<u>Scale</u>	<u>Datum</u> <sup>1</sup>	<u>File No.</u> <sup>2</sup>
1878 <sup>3</sup>	1:9600	MLLW	C-949 - C 951
1934 <sup>3</sup>	1:9600	MLLW	C-949 - C-951
1937 <sup>3</sup>	1:9600	MLLW	C-949 - C-951
1949 <sup>3</sup>	1:9600	MLLW	C-949 - C-951
1958 <sup>3</sup>	1:9600	MLLW	C-949 - C-951
Jun-Aug 1963	1:3600	MLLW	902-B - 907-B
1967 <sup>3</sup>	1:9600	MLLW	C-949 - C-951
Apr 1969	1:9600	MLLW	C-921 - C-923
Apr 1970	1:4800	MLLW	C-926-70-4 - C-931-70-4
Dec 1982 - Jan 1983	1:4800	MLLW	E-906 - E-910

1) MLLW - Mean Lower Low Water.

2) U.S. Army Corps of Engineers, Los Angeles District file numbers.

3) Month of survey not available.

Historic changes in shoreline position exhibited consistent trends along shoreline Segments 2 and 3. The southern region of shoreline between the Santa Ana River and Huntington Pier experienced shoreline progradation for every measured time interval except that between 1958 and 1963. The northern coastal segment from Huntington Pier to Anaheim Bay experienced both erosion and accretion; however, shoreline progradation was dominant between 1934 and 1983. A more comprehensive discussion of historical shoreline changes in this region is provided by Gravens (1988) and Signal Landmark (1988). Finally, the shoreline position data for the years 1963, 1970, and 1983 were assembled at 200-ft intervals (the cell spacing used in the shoreline change simulations) for use in the shoreline change model.

#### Wave Data

Three parameters are used by both the shoreline change model and the nearshore wave transformation model to describe the characteristics of the wave climate. These are the significant wave height, dominant wave period, and incident wave angle.

Four sources of wave data were available for application to the project coast during the conduct of the preliminary shoreline response study. These are the Marine Advisors (MA) (1961) hindcast, the National Marine Consultants (NMC) (1960), hindcast, two U.S. Army Corps of Engineers Littoral Environment Observation (LEO) Stations (Sherlock and Szuwalski 1987), and a slope array wave gage maintained by Scripps Institution of Oceanography (SIO).

The NMC and MA hindcasts cover the years 1956, 1957, and 1958, and provide percent occurrences for given deepwater wave heights and periods. Since the shoreline change model requires a time series of input wave conditions, the hindcast wave data were used for statistical comparison purposes only. The LEO program had two stations on the project coast, at Bolsa Chica and Huntington Beach. LEO data are available for the Bolsa Chica station from October 1979 to May 1982, and for the Huntington Beach station from October 1979 to April 1985. The LEO program provides daily visual estimates of the breaking wave height, angle, and period, as well as other littoral environment data. A 1-year time history of wave data was selected from each of the LEO stations for use in the comparison of available wave data.

As part of the Coastal Data Information Program sponsored by the U.S. Army Corps of Engineers and the California Department of Boating and Waterways, SIO maintains a slope array wave gage in water approximately 26.9 ft deep just offshore of Bolsa Chica (SIO reports the gage depth as 8.2 m). This wave gage has been in place since November 1980, and the longest period of continuous data 27 months from February 1981 to May 1983. The next longest continuous record contains 14 months of data (June 1986 to August 1987). These two continuous records were combined to obtain a continuous 3-year time history of significant wave height, incident angle, and wave period at 6-hour intervals. The 3-year time history of wave conditions was compiled in a manner that preserved the normal progression of the seasons.

The next step in the examination of the wave data was to compare the statistics of the available data sets at the stations of interest (MA hindcast Station B, NMC hindcast Station 7, two LEO stations (Bolsa Chica and Huntington Beach), and the SIO wave gage at Sunset Beach). Because the shoreline change model uses a time-step procedure to calculate shoreline change, only the LEO data and the gage data could be easily adapted. The gage data set was the preferred data set because it provides a 17-minute statistical and objective summary at 6-hour intervals (the time step typically used in the shoreline change model). Alternatively, the more approximate and subjective LEO data sets could have been used, but the observed wave conditions would have been required to be assumed to persist for the entire day (4 time steps). Therefore, the intent of the comparison was to verify the preferred data set (the SIO gage data) in terms of representative wave statistics.

The wave data for the five stations were transformed to a depth of 26.9 ft (the depth of the SIO gage) using linear wave theory refraction and shoaling in order to compare the distribution of incident wave angles between the data sets. Wave roses of incident angles were computed for each of the stations. Comparison of the SIO gage data with the two LEO stations and MA Station B hindcast revealed a distinct reduction in the variability of incident wave angles in the gage data. In fact, the gage data show nearly twice the percentage of waves occurring in the southwest (directly offshore direction) angle band than any of the other stations. Additionally, the LEO stations and MA Station B hindcast data sets have approximately 15 percent

more waves approaching from northern angle bands than from southern angle bands, whereas the SIO gage data show less than 10 percent more waves from this direction. The average monthly incident wave angle and standard deviation,  $\sigma$ , of the wave angle were calculated for the LEO stations and the gage data. The results are presented in Table 2, and confirm that the variability of the gage data is much less than that of the LEO stations. Also, the LEO station data sets each contain only 3 months with positive average incident wave angles (waves approaching from the south), whereas the gage data have 6 months with a positive average angle. These comparisons led to the conclusion that the incident wave angles would probably require adjustment during calibration of the shoreline change model if the SIO gage data set was used.

Table 2  
Representative Average Monthly Incident Wave Angles<sup>1</sup>

Month	LEO; Bolsa Chica		LEO; Huntington Bch.		SIO gage	
	(deg)	$\sigma^2$	(deg)	$\sigma^2$	(deg)	$\sigma^2$
January	-9.1	6.4	-24.5	10.4	-3.8	3.2
February	-3.8	7.4	-11.2	11.1	-5.1	2.8
March	-18.5	12.4	-6.4	11.3	-4.7	3.4
April	-17.8	9.1	-16.5	11.5	-3.4	4.3
May	-16.4	13.5	-1.2	11.7	0.6	4.2
June	-11.0	16.4	3.9	9.4	1.7	4.2
July	1.0	14.3	25.8	11.9	3.3	4.5
August	6.2	13.0	-4.6	13.5	3.4	3.9
September	13.9	13.2	-1.4	9.3	2.6	4.0
October	-4.7	9.5	4.7	12.5	2.5	3.7
November	-8.8	11.2	-9.2	12.5	-1.8	5.3
December	-9.3	5.6	-12.0	17.6	-5.1	3.3

1) Data transformed to 26.9-ft depth.

2) Symbol  $\sigma$  represents standard deviation of wave angle (deg).

The distributions of wave period and wave height were calculated for the LEO stations, the MA hindcast, and the gage data. All four data sources show similar distributions of wave period. The calculated distributions of wave height for the LEO stations and the gage data indicate that the gage data have a greater percentage of larger wave heights than the LEO stations; however, the distribution of the wave heights for the three data sources are similar.

Based on these comparisons of the available wave data for the project site, a decision was made to use the SIO gage data as input to the shoreline

change model in the preliminary study. However, the SIO gage provides a weighted angle defined from components of the wave radiation stress, and not a direct wave angle. Therefore, direction results must be interpreted with caution. Although the distribution of incident wave angles did not compare well with the other data sources, a uniform adjustment to the incident wave angles was determined during the model calibration.

#### WAVE TRANSFORMATION ANALYSIS

Because the magnitude and direction of the longshore sand transport rate are dependant on the sine of the breaking wave angle with respect to the shore and on the breaking wave height raised to the  $5/2$  power, calculated shoreline change is sensitive to the input wave conditions. In order to obtain accurate estimates of the nearshore wave climatology, a wave transformation model is required which accounts for wave refraction, diffraction, and shoaling over a natural bathymetry. The numerical model, Regional Coastal Processes WAVE (RCPWAVE) (Ebersole, Cialone, and Prater, 1986), was utilized to calculate the propagation of representative classes of linear waves over a digitized bathymetry which extended from the east jetty of Anaheim Bay to beyond the Santa Ana River. RCPWAVE accounts for refraction, shoaling, and diffraction caused by the underlying bathymetry and can be applied on a regional basis economically. The bathymetric grid used in this study consisted of 97 cells alongshore and 22 cells offshore, and grid cell dimensions were 600 ft alongshore and 300 ft offshore.

Execution of the wave transformation model for every offshore wave condition would require extensive resources and would not be justified considering the level of accuracy and sophistication of the data input and numerical models. Therefore, another approach was taken which is commonly used in regional-scale shoreline response studies performed by CERC (Kraus et al. 1988). In preparatory analysis, the offshore wave data were separated into seven 22.5-deg angle bands and two 12.25-deg angle bands centered about the compass directions of northwest, west northwest, west, etc. An RCPWAVE run was performed for wave periods from 5 sec to 21 sec in 2-sec increments in each angle band. A wave height of unity and a period corresponding to wave periods existing in the offshore wave data was input at the offshore boundary (at a depth approximately equal to that of the measured or predicted offshore



wave data) of the computational grid at an incident angle equal to the central angle of the angle band. The model then calculated the wave transformation over the actual bathymetry to the wave break point. The results (wave height transformation coefficient and nearshore incident wave angle) were saved at grid points alongshore at a nominal depth of 15 ft. The results were written to a data base and keyed to the input angle band and wave period. This allowed the shoreline change model to read the offshore wave conditions at a specific time step and calculate a key based on the incident wave angle and wave period. The key was then used to identify the corresponding nearshore wave conditions along the project coast. Using this methodology, nearshore wave heights and incident angles were obtained at 600-ft intervals for input to the shoreline change model. The use of RCPWAVE in this manner allowed the shoreline change model to account for major bathymetric features offshore which may cause convergence or divergence of wave energy along the coast.

#### SHORELINE CHANGE MODEL

The acronym GENESIS stands for GENeralized model for Simulating Shore-line change. A detailed description of the model is provided in Hanson (1987, 1989) and Hanson and Kraus (1989). GENESIS is a generalized system of numerical models and computer subroutines which allow simulation of long-term shoreline change under a wide variety of user-specified conditions.

GENESIS calculates local wave breaking, longshore sand transport rate, and the resulting plan shape evolution of the modeled coast. The effect of natural features such as sea cliffs, and coastal structures and activities such as seawalls, groins, and beach fills are incorporated in the model by modification of the transport rate through boundary conditions and constraints. The diffraction effect of detached breakwaters and long groins on the local wave climate is represented around and behind these structures. Kraus (1989) describes the capabilities and limitations of the model.

GENESIS can be utilized with two types of wave inputs depending on the available data and degree of computational effort required. A single offshore or deepwater wave condition can be input, and the wave model within GENESIS will calculate breaking wave conditions along the modeled reach. Alternatively, a more sophisticated wave transformation model which describes wave propagation over the actual offshore bathymetry (such as RCPWAVE) can be used

to perform the required nearshore wave transformation. In this case, GENESIS retrieves the nearshore wave characteristics (output from RCPWAVE) from a data base and performs local refraction, diffraction, and shoaling calculations to obtain a breaking wave height and angle at intervals alongshore. In either case, once the breaking wave field along the modeled reach is available, longshore sand transport rates are calculated and the shoreline positions updated.

GENESIS is primarily used to calculate long-term changes in shoreline position caused by the alongshore movement of sand. Offshore transport of sand caused by, for example, intense short-duration storm events, is not modeled. However, estimates of shoreline changes resulting from these events could be superimposed on the shoreline position calculated by GENESIS to obtain a first approximation of the potential variation about the calculated shoreline position. Calculations of short-term storm-induced beach change are given by Kraus and Larson (1989) and Scheffner (1989).

#### **PRELIMINARY SHORELINE RESPONSE STUDY**

In the preliminary shoreline response study, the numerical model GENESIS was calibrated for the period July 1963 to April 1970 and verified for the period April 1970 to January 1983, for the project coast. The SIO wave gage data set and the shoreline position data described above provided the primary inputs to the model. Simulations of the potential shoreline responses to the proposed inlet entrance system were then performed. This study was preliminary since the wave data were limited (3-years long), and the estimates were of sufficient accuracy to determine only the general magnitude of impacts so that conclusions could be reached regarding the ability to mitigate impacts to a prescribed level. Six design alternatives, and several variations of the general alternatives, were modeled using GENESIS. The intent of the simulations was to estimate the shoreline impacts of the proposed Bolsa Chica navigable ocean entrance system. A summary of the six modeled design alternatives is given in Table 3.

GENESIS was configured for application to the project coast. The model reach extends from the east jetty of Anaheim Bay to the north jetty of the Santa Ana River and has 270 200-ft wide alongshore calculation cells. The southern boundary condition at the Santa Ana River was simulated as a short

groin. The implication of this boundary condition is that a portion of the calculated sand at the boundary can pass into or out of the modeled reach, provided that the calculated maximum depth of longshore transport at the given time step exceeds the 3-ft depth at the tip of the jetty. The northern boundary condition at the east jetty of Anaheim Bay was simulated as a long

Table 3  
Summary of Modeled Design Alternatives (Preliminary Study)

<u>Design Alternative No.</u>	<u>Entrance Channel Location and Width</u>	<u>Detached Breakwater</u>	<u>Sand Management</u>
1 <sup>1</sup>	Proposed Site, 800 ft	per SLC design	No
2	Proposed Site, 1000 ft	per SLC design	No
3	Warner Avenue, 800 ft	per SLC design	No
4	South of Site, 800 ft	per SLC design	No
5	Proposed Site, 800 ft	Dog-leg breakwater attached to north jetty	No
6	Proposed Site, 800 ft	per SLC design	Yes <sup>2</sup>

- 1) Design Alternative No. 1 was simulated five times with the following variations in the input wave conditions:
  - Run 1a. Without modification of input wave conditions.
  - Run 1b. Increased wave height by 15%.
  - Run 1c. Decreased wave height by 15%.
  - Run 1d. Wave angle rotated 10 deg south (counter-clockwise).
  - Run 1e. Wave angle rotated 10 deg north (clockwise).
- 2) Design Alternative No. 6 was run seven times:
  - Run 6a. Without modification of input wave conditions, 1 million cu yd feeder beach down drift of inlet.
  - Run 6b. Wave angle rotated 10 deg north (clockwise), 1.5 million cu yd feeder beach down drift of inlet.
  - Run 6c. Wave angle rotated 10 deg north (clockwise), 1.5 million cu yd feeder beach down drift of inlet.
  - Run 6d. Without modification of input wave conditions, 1 million cu yd feeder beach down drift of inlet, 2.65 million cu yd feeder beach at Surfside-Sunset.
  - Run 6e. Without modification of input wave condition, 1 million cu yd feeder beach down drift of inlet, 4.65 million cu yd feeder beach at Surfside-Sunset.
  - Run 6f. Wave angle rotated 10 deg. north (clockwise), 1.5 million cu yd feeder beach down drift of inlet, 4.65 million cu yd feeder beach at Surfside-Sunset.
  - Run 6g. Wave angle rotated 10 deg. north (clockwise), 2 million cu yd feeder beach down drift of inlet, 4.65 million cu yd feeder beach at Surfside-Sunset.

non-diffracting jetty. The implication of this boundary condition is that no sand can move into the modeled reach from the north. Two other constraints were imposed inside the modeled reach. These were the Huntington Beach Pier and the sea cliffs located between the proposed ocean entrance at Bolsa Chica and the Huntington Beach Pier. The Huntington Beach Pier was simulated as a permeable groin (Hanson 1988, 1989; Gravens and Kraus 1989). The permeability factor was determined during the model calibration. The sea cliffs along the Huntington Mesa were simulated as a seawall. This boundary condition prohibits the shoreline from eroding beyond the present position of the cliffs.

In all the design alternative simulations, the 1983 surveyed shoreline position was used as the initial shoreline. The design alternative simulations were performed for 5- and 10-year prediction periods using the same 3-year-long time history of wave conditions as employed in the calibration, repeated as necessary. Calculations were also performed in which the wave input were varied to establish a range of potential shoreline changes, in recognition of large variability in the incident wave climate (Gravens 1988, Gravens 1990). In the design alternative simulations, sand transport into the proposed entrance channel (between the jetties) was permitted but transport out was not. Thus, the ocean entrance channel was modeled as a sediment sink.

#### COMPREHENSIVE SHORELINE RESPONSE

The comprehensive shoreline response modeling consisted of repeating the analysis of the preliminary shoreline response task, except that input wave conditions were derived from the hindcast wave estimates at stations located near the lateral boundaries of the modeled shoreline reach. This allowed systematic variations in the incident waves (wave height and wave angle variations along the shore) to be accounted for in the wave transformation model RCPWAVE (Gravens 1990). The hindcast wave estimates were transformed from the hindcast stations to the offshore boundary of the RCPWAVE grid as illustrated in Figure 2. This transformation included the shadowing effect of Point Fermin at hindcast Station 14 and the local contour orientations at both of the hindcast Stations. RCPWAVE simulations were then performed, and the calculated wave height and angle gradients were imposed as offshore boundary conditions within the model.

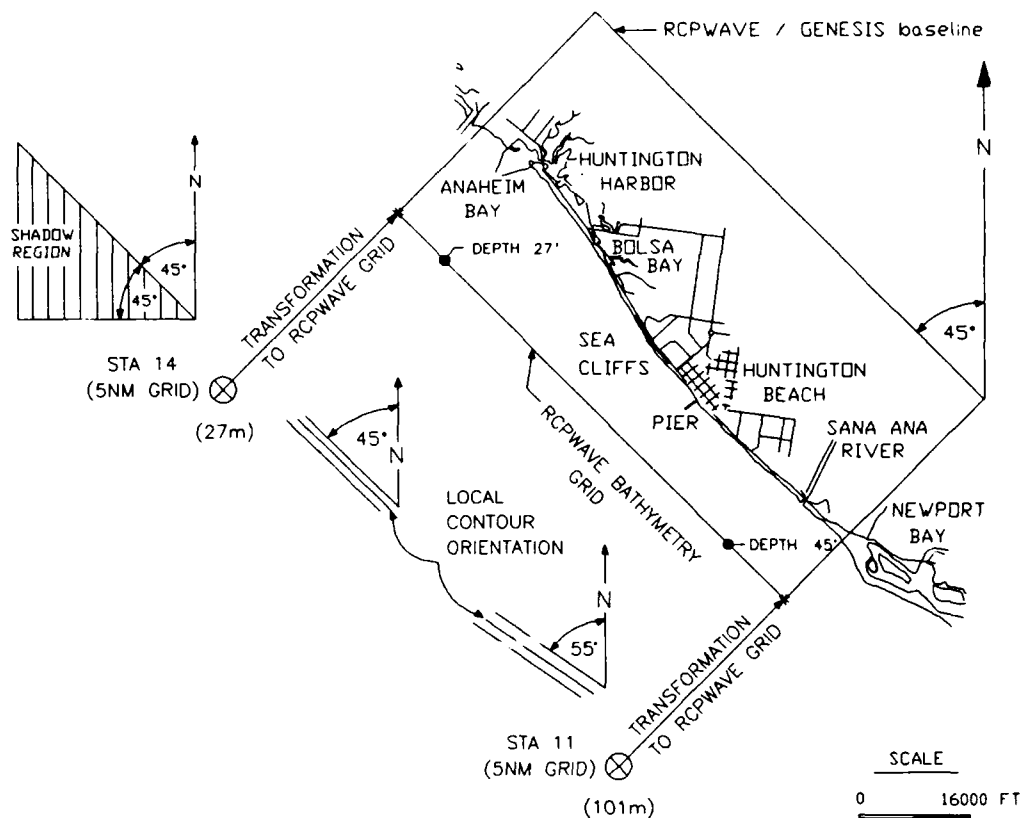


Figure 2. Wave transformation hindcast stations to RCPWAVE grid

After transforming the hindcast wave estimates, potential longshore sand transport rates were calculated for the various shoreline orientations within the project reach. These potential transport rates are presented in the form of a total littoral drift rose (Walton and Dean 1973) in Figure 3. Three curves are given in Figure 3. The curve with the circular symbols represents the average downcoast littoral drift for the 20-year northern hemisphere hindcast of sea and swell wave conditions and the curves with the "x" and triangular symbols represent the average upcoast littoral drift for the available two years of southern hemisphere swell wave estimates. It is interesting to note that there is a reversal in the direction of the average net longshore littoral drift and that this reversal occurs at different shoreline orientations depending on the time series of southern swell wave conditions used in the calculation.

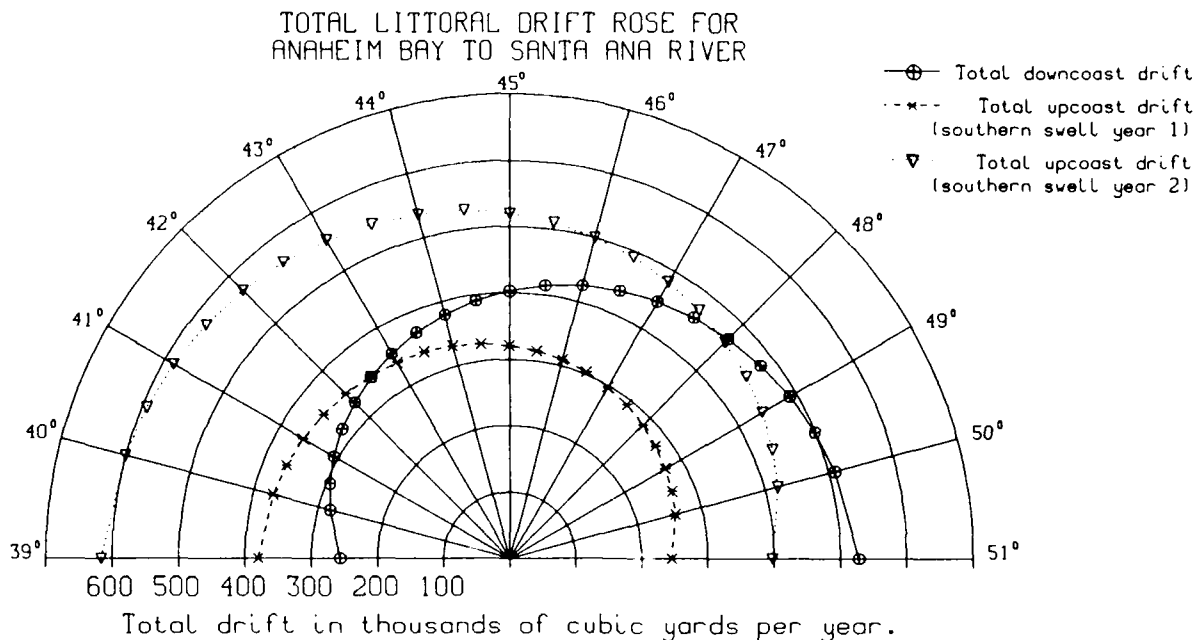


Figure 3. Total littoral drift rose, Anaheim Bay to Santa Ana River

#### Model Calibration and Verification

The shoreline change model was then calibrated and verified using the same boundary conditions and constraints as were imposed in the preliminary study expect that the transformed hindcast wave estimates were used for wave input rather than the SIO wave gage data. The results of the final model calibration simulation are presented in Figure 4. With regard to the calibration three general observations are noted. First, in the Anaheim Bay entrance area (between alongshore coordinates 220 and 260), there are significant differences between the calculated and measured shoreline positions. These differences are due in part to the reflection of waves from the east Anaheim Bay jetty (a process which was not modeled) and to a massive (4 million cu yd) renourishment of the Surfside-Sunset feeder beach in 1964. The percentage of fine material contained in the beach fill is unknown; consequently, the initial losses of fill material could not be estimated or accounted for in the model. Model results in this region should be viewed with caution.

Secondly, in the vicinity of the Huntington Pier (between alongshore coordinates 80 and 90), it is noted that the predicted shoreline positions do not agree well with the survey. The lack of agreement is due to limitations in the groin boundary condition used to simulate the effect of the pier. A

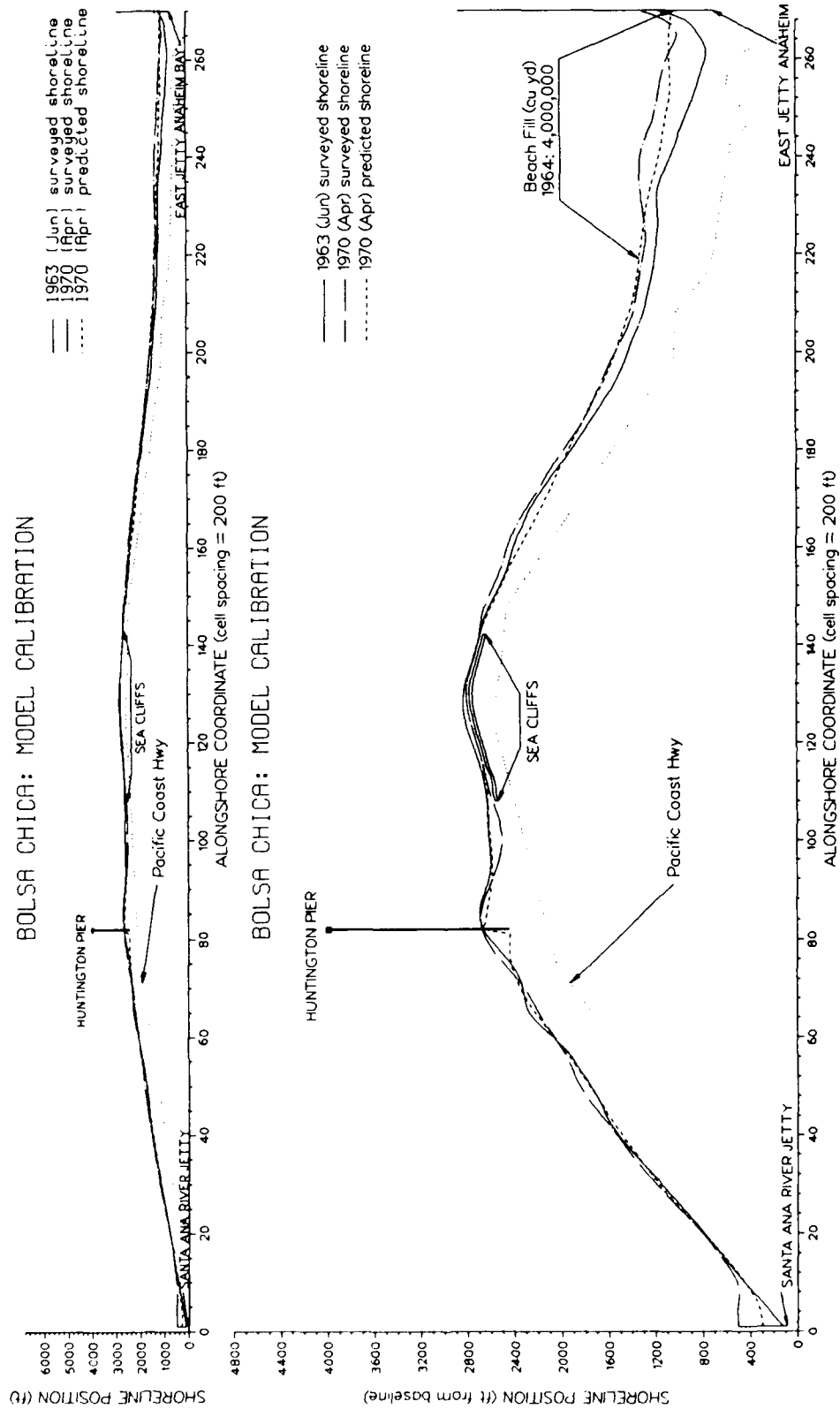


Figure 4. Model calibration results

detailed investigation of the imposed boundary condition at the pier was performed (Gravens 1990), and the conclusion was that the boundary condition imposed at the Huntington Pier had no significant effect on the model results northeast of the sea cliffs over the modeling interval.

Finally, in the vicinity of the proposed entrance system (between alongshore coordinates 155 and 220), the predicted and measured shoreline positions are in very good agreement. Model results for this region are considered to have high reliability.

Average annual net and gross longshore sand transport rates calculated by the model are presented in Figure 5. Also given in Figure 5 are the net transport rates for the year which produced the maximum southerly and maximum northerly littoral drift rates. These curves are presented to point out the profound variability in both the direction and magnitude of the net longshore sand transport rate which can be encountered from year to year depending on the incident wave climate. It should be noted that there is a reversal in the average annual net longshore transport direction in the vicinity of the proposed entrance system. This unique feature is very important with regard to the proposed entrance system impacts on adjacent shorelines, indicating that the entrance would have minimal adverse impact on adjacent shorelines.

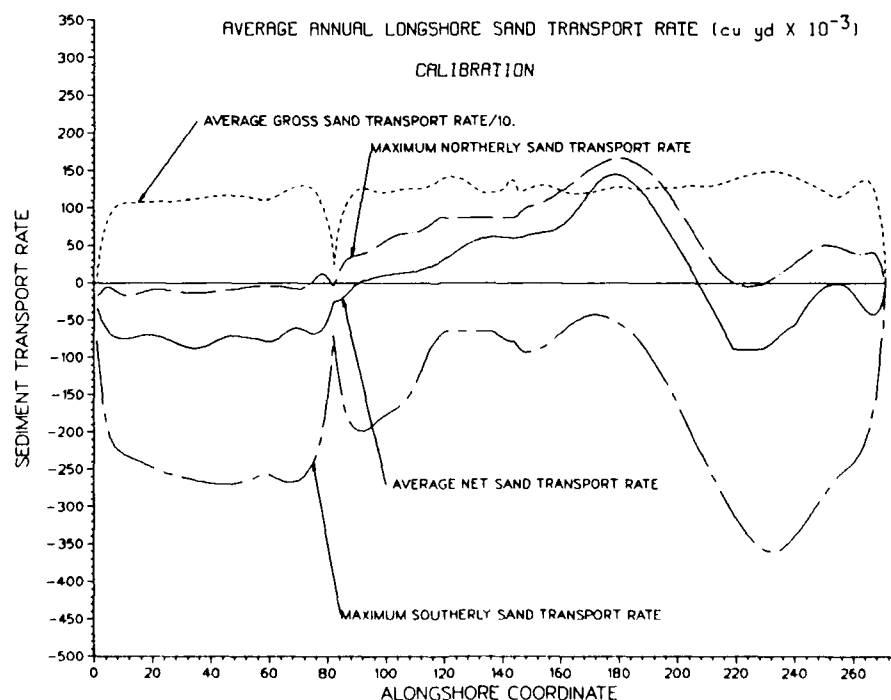


Figure 5. Average annual longshore sand transport rates (1963-1970)



## Model Tests

After calibrating and verifying the shoreline change model, eight conceptual design alternatives were modeled, and several simulation variations were performed for each of the alternatives. The intent of the simulations was to quantify the shoreline impacts of the proposed Bolsa Chica navigable ocean entrance system. In the simulation of Alternatives 1 and 3, no sand management activities were specified; in other words, there were no inputs of beach nourishment material along the modeled reach. In the simulation of Alternatives 2, 4, 5, 6, and 8, renourishment of the Surfside-Sunset feeder beach was specified at 1 million cu yd every 5 years. In the simulation of Alternatives 7 and 8, impact mitigation sand management techniques were modeled. A summary of the eight design alternatives evaluated is given in Table 4.

Table 4

Summary of Modeled Design Alternatives (Comprehensive Study)<sup>1</sup>

<u>Design Alternative No. &amp; Simulation Code</u>	<u>Entrance Channel Location and Width</u>	<u>Surfside-Sunset Feeder Beach</u>	<u>Impact Mitigation Sand Management</u>
1 WP1A,WP1B,WP1C	Without Project	No	No
2 WP2A,WP2B,WP2C	Without Project	Yes	No
3 PRO1A,PRO1B,PRO1C	Proposed Site, 800 ft	No	No
4 PRO2A,PRO2B,PRO2C	Proposed Site, 800 ft	Yes	No
5 PUC2A,PUC2B,PUC2C	Warner Avenue, 800 ft	Yes	No
6 PDC2A,PDC2B,PDC2C	South of Site, 800 ft	Yes	No
7 SM1A,SM1B,SM1C	Proposed Site, 800 ft	No	Yes
8 SM2A,SM2B,SM2C	Proposed Site, 800 ft	Yes	Yes

- 1) Design Alternatives 1 through 8 were simulated three times to investigate the effect of potential variabilities in the incident wave climate as follows:
- Alternating available southern swell wave conditions (years 1 and 2).
  - Low-intensity southern swell wave conditions (year 1).
  - High-intensity southern swell wave conditions (year 2).

As in the preliminary study, the 1983 surveyed shoreline position was used as the initial shoreline. All model tests were performed for 5- and 10-year simulation (prediction) periods using the same randomly selected 10-year time history of northern hemisphere sea and swell wave conditions. The southern hemisphere swell component of the incident wave climate was varied

depending on the particular model simulation, as shown in Table 4. The model simulations were performed assuming that the proposed entrance channel and detached breakwater were constructed in 1983. Hence, the predicted 1988 and 1993 shoreline positions represent the expected shoreline positions after 5 and 10 years. As in the preliminary study, the ocean entrance channel was again modeled as a sand sink.

The model results of design Alternative 2, simulation code WP2A (without-project design alternative), are given in Figures 6 and 7. Figure 6 shows the predicted 5- and 10-year shoreline positions as well as the seaward-most and landward-most calculated shoreline positions, whereas Figure 7 shows the calculated average annual net and gross longshore sand transport rates. Again, a reversal in the average net longshore transport direction in the vicinity of the proposed entrance system occurs.

The model results of design Alternative 4, simulation code PRO2A (preferred alternative), are presented in Figures 8 and 9. Figure 8 shows the predicted shoreline positions in which shoreline progradation is observed on both sides of the entrance system. The data in Figure 9 provides the explanation for progradation of shorelines adjacent to the proposed entrance system. The average net longshore sand transport rates northwest of the entrance are negative, indicating sand transport in a southeasterly direction, whereas on the southeast side of the entrance the transport rates are positive indicating northwesterly sand transport. Net and gross longshore sand transport rates at the jetties of the proposed entrance are zero. This interruption of the littoral drift in a region of converging net longshore transport directions results in sand accumulation on both sides of the entrance.

In order to isolate the shoreline impacts directly attributable to the proposed navigable ocean entrance system, the results of the without-project simulations (Alternatives 1 and 2) were compared to the results of the preferred alternative simulations (Alternatives 3 and 4). The comparisons were made based on shoreline change from the 1983 surveyed shoreline positions. Figure 10 shows the shoreline change from the initial (Jan 1983) shoreline position to the predicted 10-year (Jan 1993) shoreline position for alternatives 2 and 4 presented in Figures 6 and 8.

In all the preferred alternative simulations, there is a narrow region of shoreline accretion adjacent the entrance jetties on both sides of the

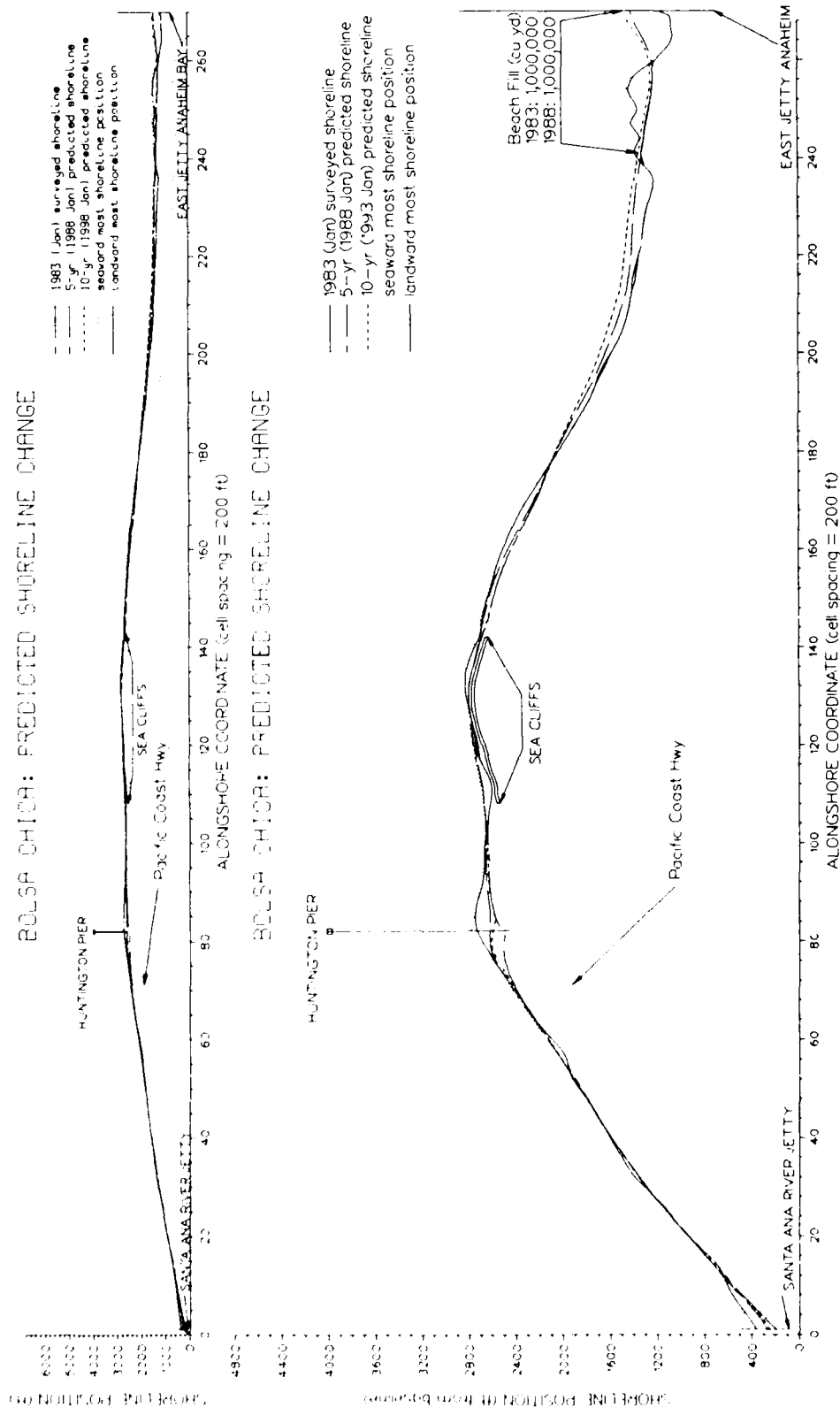


Figure 6. Alternative WP2A: without-project, with feeder beach

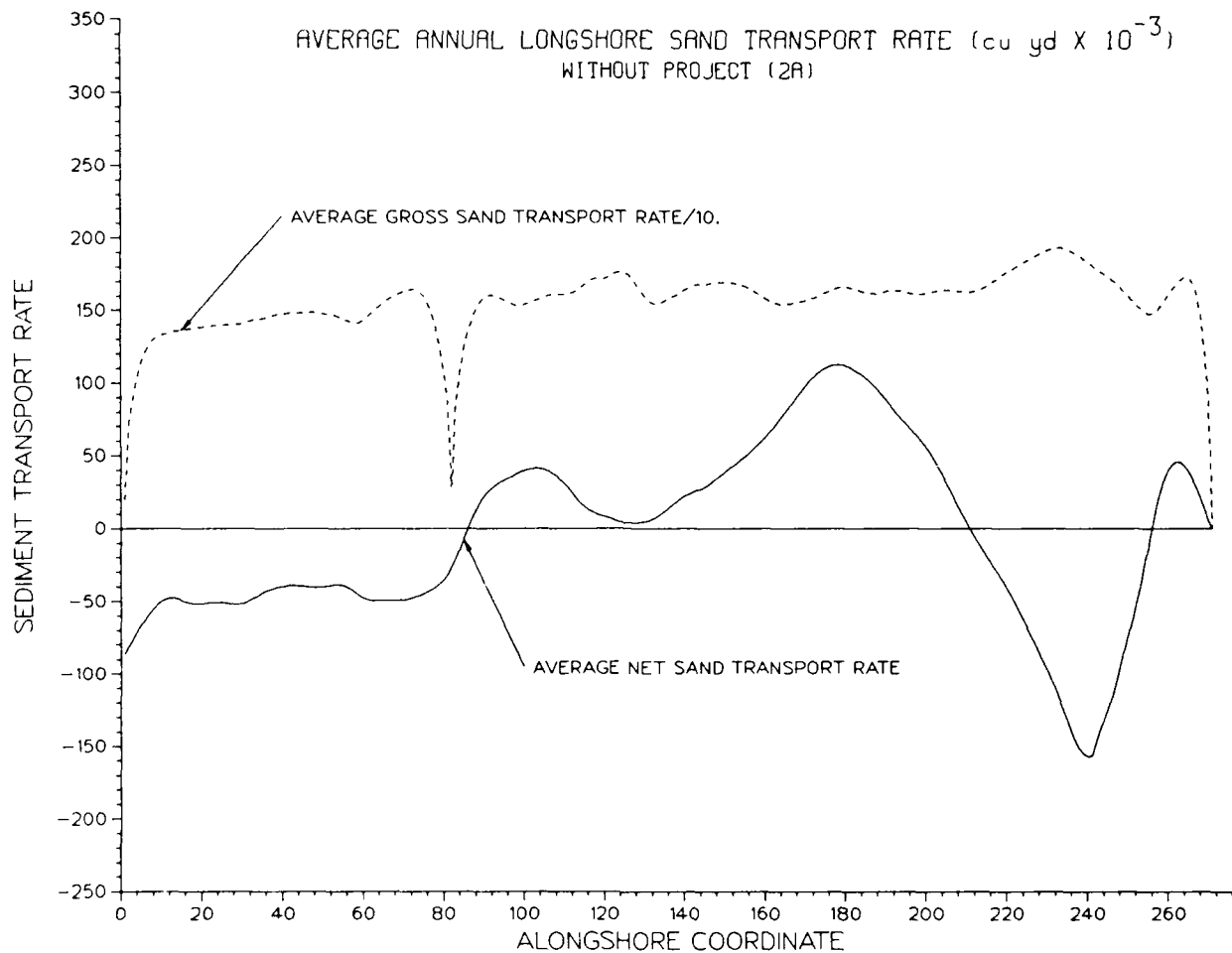


Figure 7. Alternative WP2A: average annual longshore sand transport rates

proposed channel. This region of accretion is followed by a wider zone of shoreline erosion further away from the entrance system. On the southeast side of the proposed entrance system, the alongshore width of the accretive beach varies from between 1400 ft and 2800 ft. The maximum berm width of the accretive beach occurs immediately adjacent to the jetty and varies from between 400 ft and 700 ft. On the northwest side of the entrance system, the width accretive beach varies from between 1200 ft and 2000 ft. The maximum berm width on the northwest side again occurs immediately adjacent to the jetty and varies from between 300 ft and 400 ft.

In the simulations in which year 1 of the southern swell wave conditions were used (Alternatives 3 and 4, simulation codes PR01B and PR02B, not shown in this paper; see Gravens (1990) as input to the shoreline change model the

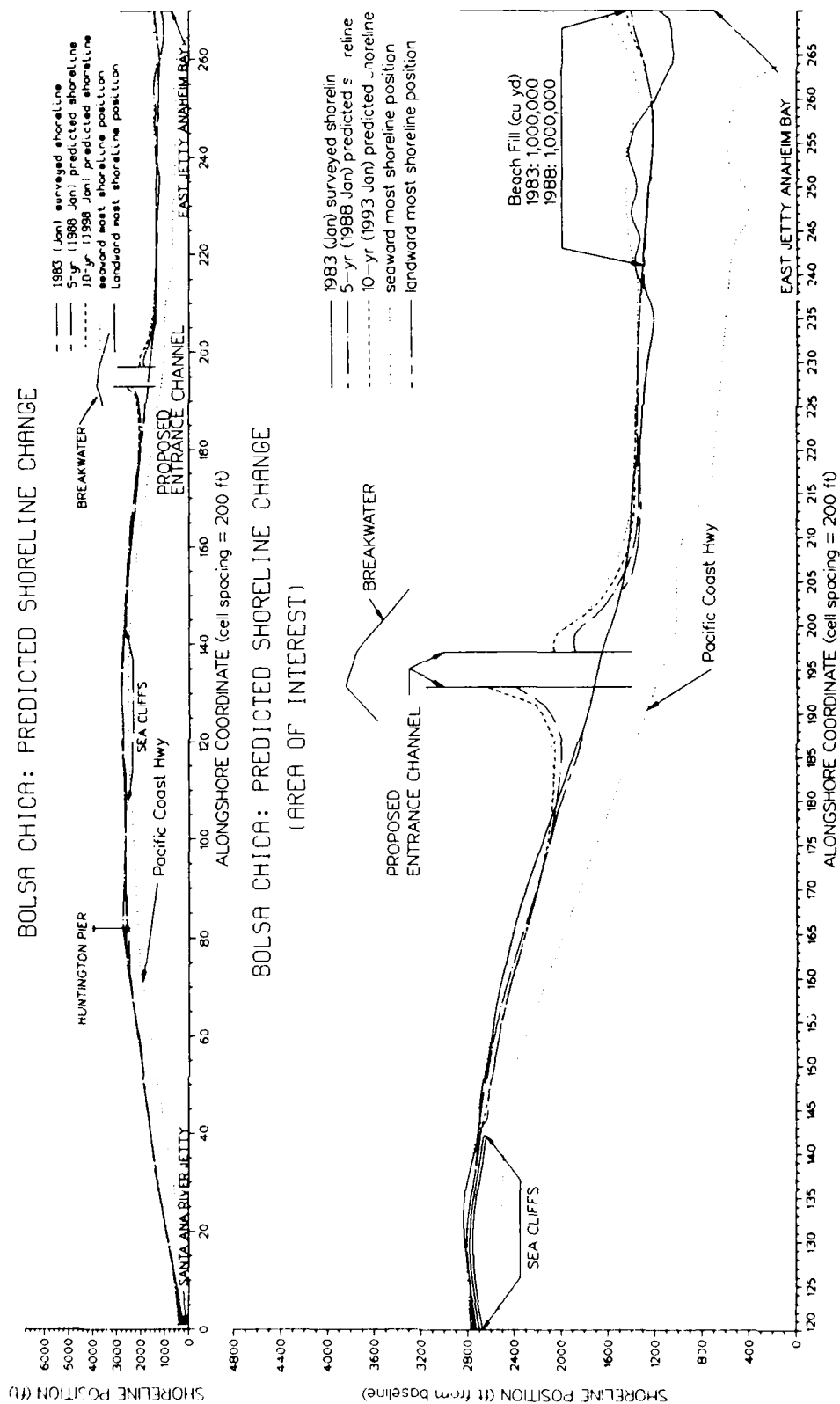


Figure 8. Alternative PRO2A: preferred alternative, with feeder beach

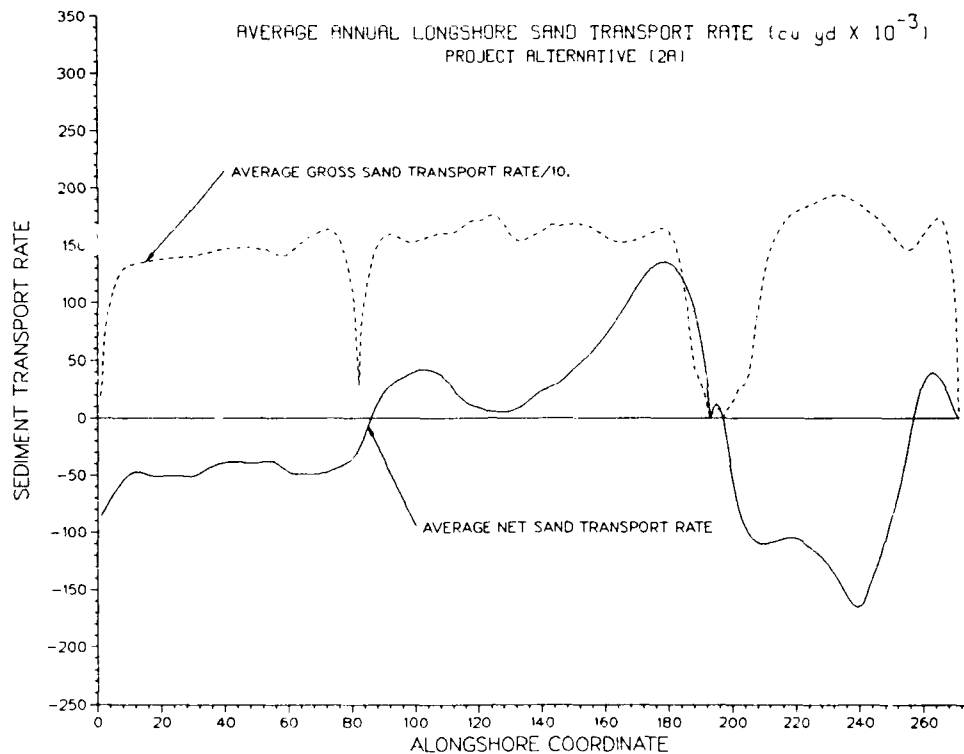


Figure 9. Alternative PRO2A: average annual longshore sand transport rates

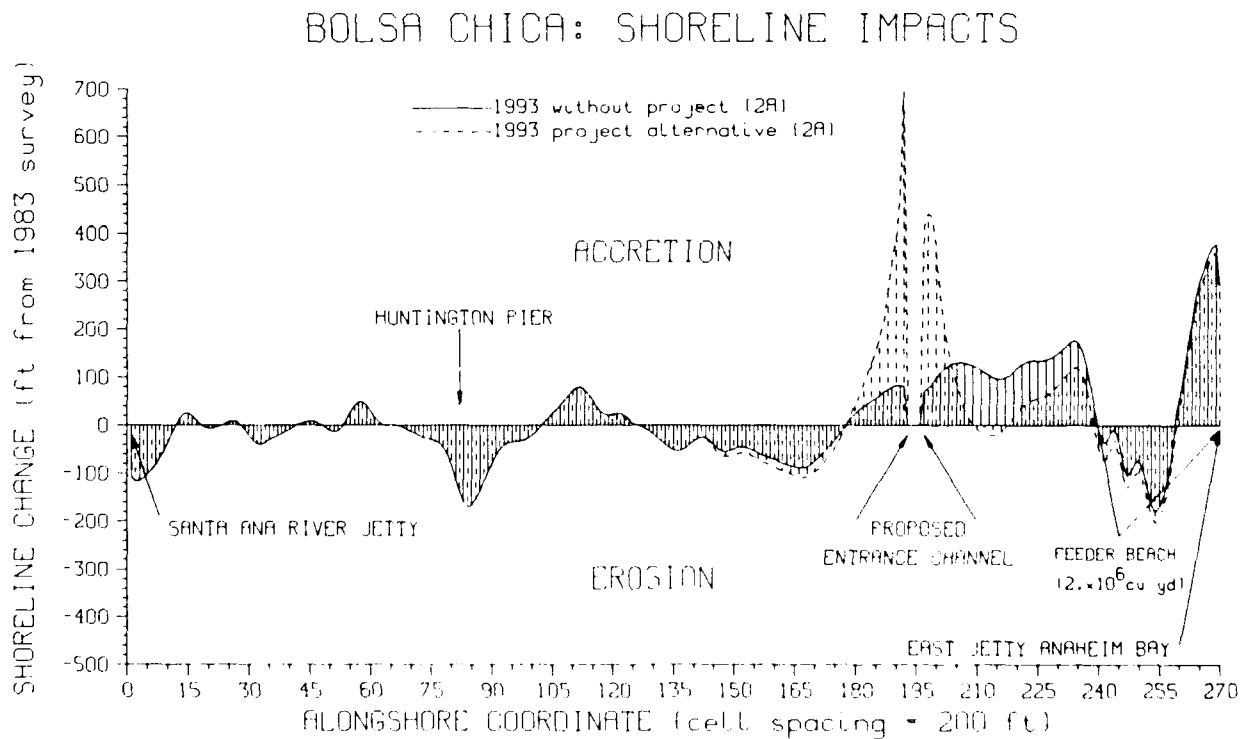


Figure 10. Predicted shoreline change from 1983 shoreline position (Alternative WP2A vs. Alternative PRO2A)

longest region of shoreline erosion on the southeast side of the entrance system was predicted. In these simulations, the alongshore width of the erosion zone is on the order of 8400 ft and within this region the shoreline is displaced about 60 ft landward.

The simulations in which year 2 of the southern swell wave conditions were used (Alternatives 3 and 4, simulation codes, PRO1C and PRO2C, not shown in this paper; see Gravens (1990) resulted in the longest extent of shoreline erosion on the northwest side of the entrance system. The predicted length of the erosion zone is on the order of 11,000 ft, and the maximum eroded berm width is about 180 ft.

The simulations in which all of the available southern swell wave conditions (year 1 and year 2) were used (shown in Figure 10), resulted in less overall shoreline erosion. The results of these simulations represent the best estimate of the expected shoreline evolution resulting from the construction of the proposed ocean entrance system at Bolsa Chica Bay. The results of the other simulations not shown in this paper (Gravens 1990) represent possible extremes about the baseline simulation given in Figure 10 and will require that impact mitigation plans (sand bypassing and/or backpassing at the entrance) be flexible.

The previous model results and analysis were presented to the SLC, which established the following two criteria for the impact mitigation simulations:

- a. Only that sand which accumulates within 1500 ft of the entrance jetties may be utilized for sand bypassing and/or sand backpassing. No new sources of sand will be used for impact mitigation sand management.
- b. A successful sand management plan will be one in which shoreline change from the 1983 surveyed shoreline position is accretive, or, if the without-project alternative indicates erosion, the sand management plan must indicate equal or less erosion.

Three different sand management plans were developed for the three different input wave data sets (the "A", "B", and "C" type simulations as indicated in the simulation code, see Table 4). In general terms, the sand management plans require that infrastructure be put in place which will be capable of: 1) backpassing approximately 300,000 cu yd/year of sediment from shorelines adjacent to both sides of the entrance system to shorelines on the order of 1/2 to 1 mile away from the entrance, and 2) bypassing approximately

150,000 cu yd/year of sediment across the entrance system both from the northwest to the southeast and vice versa.

The results of the baseline sand management simulation plan "A" (Alternative 8, simulation code SM2A) are plotted in Figure 11. The impacts of the entrance system with sand management plan "A" in place are shown in Figure 12. As depicted in Figure 12 adverse impacts on adjacent shorelines attributable to the entrance system are mitigated to the prescribed level stated above.

#### SUMMARY AND CONCLUSIONS

The shoreline change model GENESIS was utilized in conjunction with other numerical models to quantify potential shoreline impacts of constructing a structured inlet entrance system at Bolsa Chica, California. The shoreline modeling task provided estimates of gross and net longshore sand transport rates along the project reach, and allowed investigation of the technical feasibility of mitigating potentially adverse shoreline impacts resulting from the proposed entrance system.

This study of the longshore sand transport processes and shoreline response resulting from the construction of the proposed ocean entrance system at Bolsa Chica Bay has shown that mitigation of any adverse impacts on the adjacent shorelines is feasible. Based on the results of the model simulations presented above the following conclusions are made:

- a. The proposed site of the new entrance system is located in a region of converging longshore sand transport, i.e., sand is transported toward the entrance system from both upcoast and downcoast.
- b. Locating the entrance system approximately 1-mile upcoast or downcoast from the proposed site will not significantly change the estimated shoreline response.
- c. Implementation of a sand management plan and infrastructure capable of the minimum requirements stated above will allow for the mitigation of adverse shoreline impacts.
- d. The Surfside-Sunset feeder beach nourishment program must be continued in order to maintain the shoreline within 2 miles of the Anaheim Bay entrance. However, the proposed entrance system at Bolsa Chica will neither aggravate nor improve the situation.



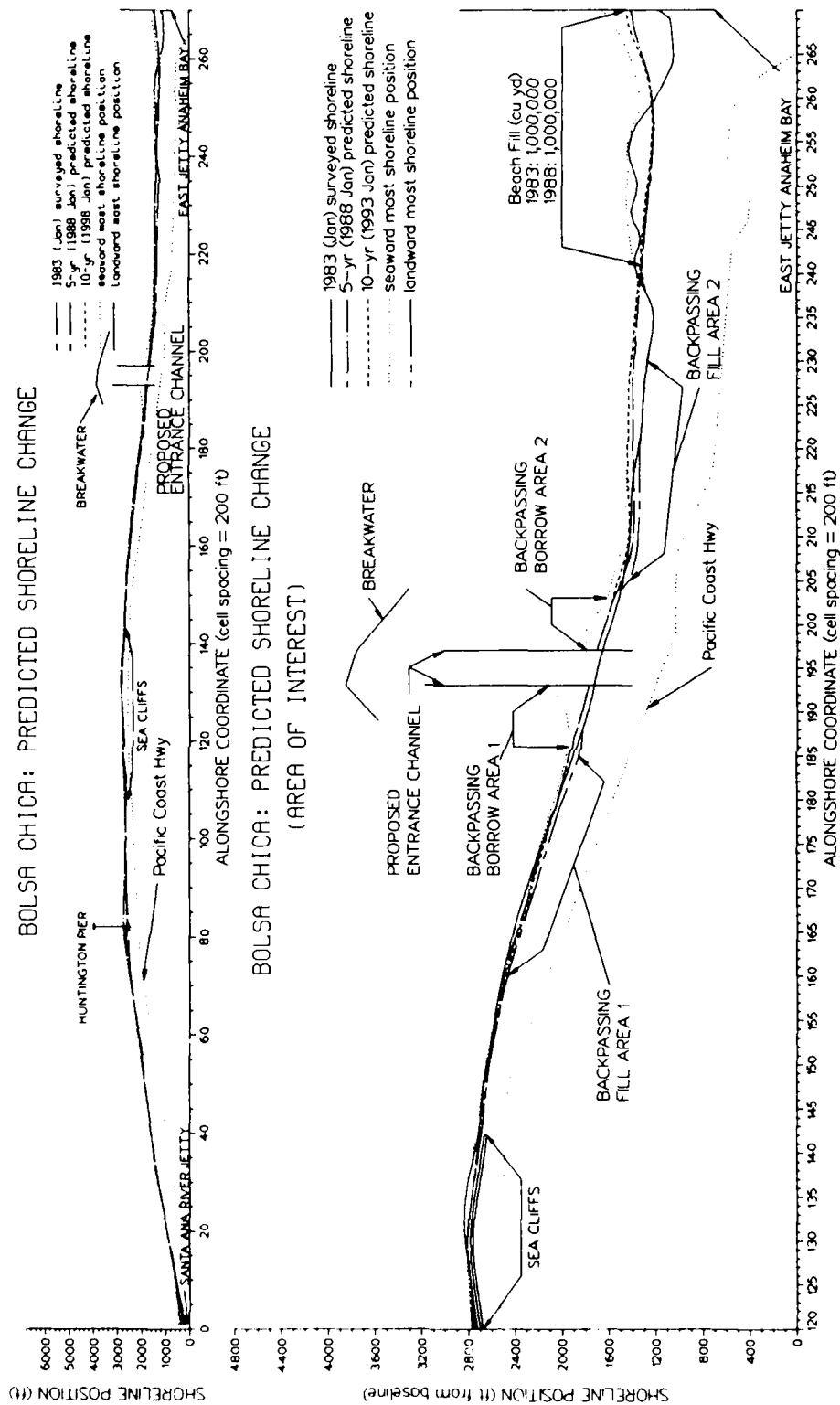


Figure 11. Alternative SM2A: sand management, with feeder beach

## BOLSA CHICA: SHORELINE IMPACTS

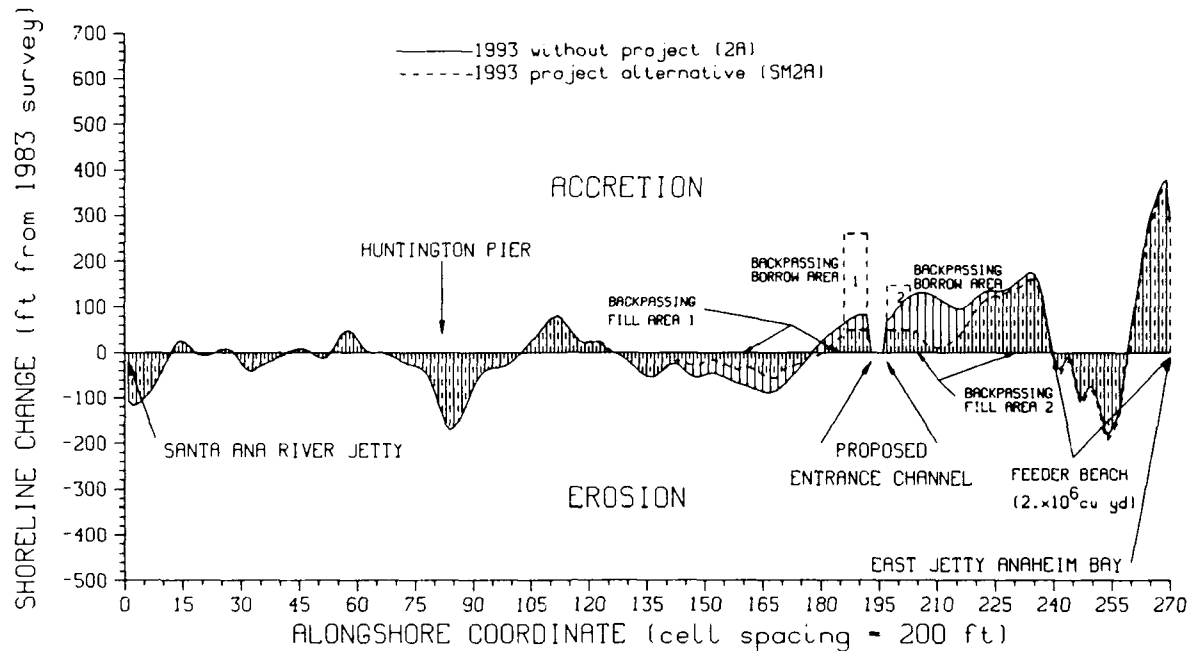


Figure 12. Predicted shoreline change from 1983 shoreline position (Alternative WP2A vs. SM2A)

### ACKNOWLEDGEMENTS

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### DISCLAIMER

Engineering services were provided to SLC by WES under authority of Title III of the Intergovernmental Cooperation Act of 1968. As such, resultant study products are based on specific technical expertise only and should not be inferred to indicate support or non-support by the Corps of Engineers for the environmental or economic aspects of any subsequent project at Bolsa Chica.

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## PREDICTION OF INITIAL PROFILE ADJUSTMENT OF NOURISHED BEACHES TO WAVE ACTION

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### ABSTRACT

A newly developed numerical model of beach profile change is applied to examine the adjustment of hypothetical beach fill designs exposed to varying waves and water level. The model calculates net cross-shore sand transport produced by breaking waves and simulates growth and movement of the main longshore bar and the berm. Evolution of an "existing" beach profile and two nourishment projects involving different fill templates is simulated over a 30-day period which includes a 3-day storm followed by a 7-day recovery period. Relative advantages and disadvantages of the templates are made evident, as well as the dependence of fill adjustment on grain size. The results demonstrate the applicability of an emerging technology for quantitative estimation of beach fill design.

### INTRODUCTION

It has been empirically and theoretically established that the average shape of the nearshore profile in equilibrium with the waves passing over it is well approximated by the power law expression (Bruun 1954, Dean 1977)

$$h = Ax^{2/3} \quad (1)$$

in which  $h$  is the water depth,  $x$  is the distance from the mean position of the shoreline (directed positive offshore), and  $A$  is a shape coefficient that is mainly a function of sand size (Moore 1982) or fall speed (Dean 1987). Equation 1 provides a simple means for estimating the ultimate (equilibrium) shape of a beach fill of given grain size and may be considered one of several "static" methods for calculating beach fill adjustment using an assumption for the final form of the profile (e.g., Edelman 1968, 1972; Vallianos 1974; Vellinga 1983).

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Static methods do not allow calculation of the temporal behavior of the profile in achieving the equilibrium shape. An engineering model to simulate time-dependent dune erosion was presented by Kriebel (1982, 1986) and Kriebel and Dean (1985), although it rests in major part on the static profile shape given by Equation 1. The model is based on a relation for the cross-shore sand transport rate expressed in terms of the breaking wave height and has been shown to produce correct orders of magnitude of foreshore and dune erosion (Birkemeier et al. 1987). A modified version of the Kriebel and Dean model has been successfully employed in engineering studies of storm-induced beach erosion and fill design on the north New Jersey coast (Kraus et al. 1988), for which a methodology was developed for calculating beach recession vs. frequency of storm occurrence relationships (Scheffner 1988). Time-dependent calculation of the rapid beach erosion accompanying a storm is expected to provide a more realistic prediction of eroded volume than a static method, since static methods pertain to a longer duration of wave action, implying a probable overestimation.

The Kriebel and Dean model includes the following simplifications: (1) independence of profile change on wave period; (2) limited capability to reproduce beach recovery on the foreshore; and (3) over-schematization of the beach profile. In contrast, beaches respond to changes in wave steepness (involving wave period), recovery may begin prior to the end of a storm (e.g., Sonu 1970, Kriebel 1987), and bars grow and move seaward during storms. Bar growth is a natural defense mechanism of the beach to break the incident waves and reduce erosive wave energy close to shore. Thus, improvements in modeling capabilities are needed for application to beach fill design.

The capability to predict the time rate of change of a beach fill adjusting to its equilibrium shape under varying waves and water level would promote an efficient project design "template" which could be used to estimate, for example, the greatest longevity of a fill of fixed total volume for a given wave climate, amortization of initial and maintenance costs vs. life expectancy, and the behavior of a fill under seasonal patterns of waves and water level where accretionary processes must also be modeled.

Recently, the authors developed a numerical model to simulate evolution of the beach profile in response to breaking waves (Larson 1988, Larson, Kraus, and Sunamura 1988). Required inputs are initial profile and fill configuration; median grain size; and time series of significant wave height, peak spectral wave period, and water level for the calculation interval. The model is compatible with Equation 1 and reproduces formation and movement of the main breakpoint bar and, to a lesser extent, the berm.

In this paper, test applications are presented to illustrate model predictions of profile adjustment to a storm event. Three representative beach profiles, an existing condition and two fills of different cross-sectional form, are subjected to a simplified and synthetic 30-day time series of waves and water level that includes a severe storm. Evolution of the existing and nourished beaches is calculated, and relative performances of the three profile configurations are compared, including dependence of fill adjustment on grain size.

## NUMERICAL MODEL

### Background

Initial model development relied on data from two independent laboratory experiment programs replicating cross-shore processes in very large wave tanks. One program was conducted by the U.S. Army Corps of Engineers (CE) in the years 1956-1957 and 1962 (Saville 1957, Kraus and Larson 1988), and the other in the early 1980s by the Central Research Institute of Electric Power Industry (CRIEPI) in Japan (Kajima et al. 1982). The combined CE and CRIEPI data set covers wave heights in the shoaling zone ranging from 0.31 to 1.80 m, wave periods from 3.1 to 16.0 sec, initial beach slopes from 1/10 to 1/50, and four median grain sizes ranging from 0.22 to 0.47 mm. In the analysis, 33 major cases in the combined data set were used, each case consisting of numerous profile surveys made under a unique combination of incident wave conditions, initial beach slope, and grain size. Kraus and Larson (1988) give a description and listing of the CE data, and Larson (1988) summarizes both the CE and CRIEPI data sets.

Because the tank studies mainly involved monochromatic waves and constant water levels, the model was further tested and refined by use of field data sets on profile change obtained by the Coastal Engineering Research Center's

Field Research Facility located at Duck, North Carolina (Howd and Birkemeier 1987). Five multi-day storm events were simulated, and model calibration parameters were examined for applicability to field conditions. For these simulations, the input consisted of measured time series of wave height, wave period, and water level, thereby greatly reducing the number of degrees of freedom in the model and increasing confidence in the calibration. The model performed well in simulating bar movement through the course of the storms (Larson, 1988; Larson, Kraus, and Sunamura, 1988).

Bars generated in the large wave tank experiments and simulated with the numerical model for constant monochromatic wave and water level conditions were much steeper than bars in the field. An important result of the field simulation was that the model successfully reproduced gentler bar slopes observed in the field, obtained under realistic conditions of time-varying wave height, wave period, and water level. The field-calibrated model was used in the present study of beach fill adjustment.

#### Wave Calculation

Transport relations used in the model require the wave height at fixed calculation points across the surf zone. Linear wave theory is applied from the seaward end of the grid, located far offshore, to the break point. Shoreward of the break point, the numerical wave simulation model of Dally (1980) is used to calculate the broken and reformed wave height.

Location of the depth-limited break point and breaking wave height are important parameters in the model. The slope of the seaward face of a bar, which changes in time as the bar grows and moves, will feed back to modify the breaking waves, since the breaker index (ratio of wave height to water depth at breaking) depends on bottom slope and wave steepness. For use in the model, the breaker index was evaluated using 121 pairs of breaking wave height/depth values from the large wave tank tests. The average breaker index was found to be 1.00, with a standard deviation of 0.25, maximum of 1.79, and minimum of 0.58. Note that the average is about 20 % greater than the commonly applied value of 0.78. The breaker index was expressed as

$$\frac{H_b}{h_b} = 1.14 \xi^{0.21} \quad (2)$$



in which  $\xi = \tan\beta/(H_o/L_o)^{1/2}$  is the surf similarity parameter,  $H_b$  is the breaking wave height,  $h_b$  is the depth at breaking,  $\tan\beta$  is the average bottom slope evaluated over a distance of one third the wavelength seaward of the break point,  $H_o/L_o$  is the wave steepness, and  $H_o$  and  $L_o$  are the deepwater wave height and wavelength. On the basis of laboratory measurements (Mimura, Otsuka, and Watanabe, 1986) and recent model tests and comparisons to field data (Larson, Kraus, and Sunamura, 1988), significant wave height should be used in the breaking wave (Equation 2) and sand transport (Equations 4-6) calculations, whereas mean wave height should be used to predict the net sand transport direction (Equation 3).

Wave height and mean water level, including setdown, setup, and runup, are calculated at each time step by using the profile shape determined from the profile change model at the previous step. In this quasi-stationary solution approach, changes in representative incident waves and bathymetry are assumed to occur on a long time scale compared to the wave period.

### Profile Change Model

#### Transport direction

Larson (1988) examined several criteria for distinguishing bar and berm formation. Net direction of cross-shore sand transport was found to be closely related to profile type, with onshore transport predominant on profiles exhibiting berm growth and offshore transport predominant if a notable bar appeared near the break point. The criterion for distinguishing profile type shown in Figure 1 was developed and is used in the model to determine net transport direction:

$$\begin{aligned} \frac{H_o}{L_o} &< C (H_o/wT)^3, & \text{erosion} \\ \frac{H_o}{L_o} &> C (H_o/wT)^3, & \text{accretion} \end{aligned} \tag{3}$$

in which  $C = 0.00070$  is an empirical coefficient,  $w$  is the sand fall speed, and  $T$  is the wave period. The parameter  $H_o/wT$  is called the dimen-

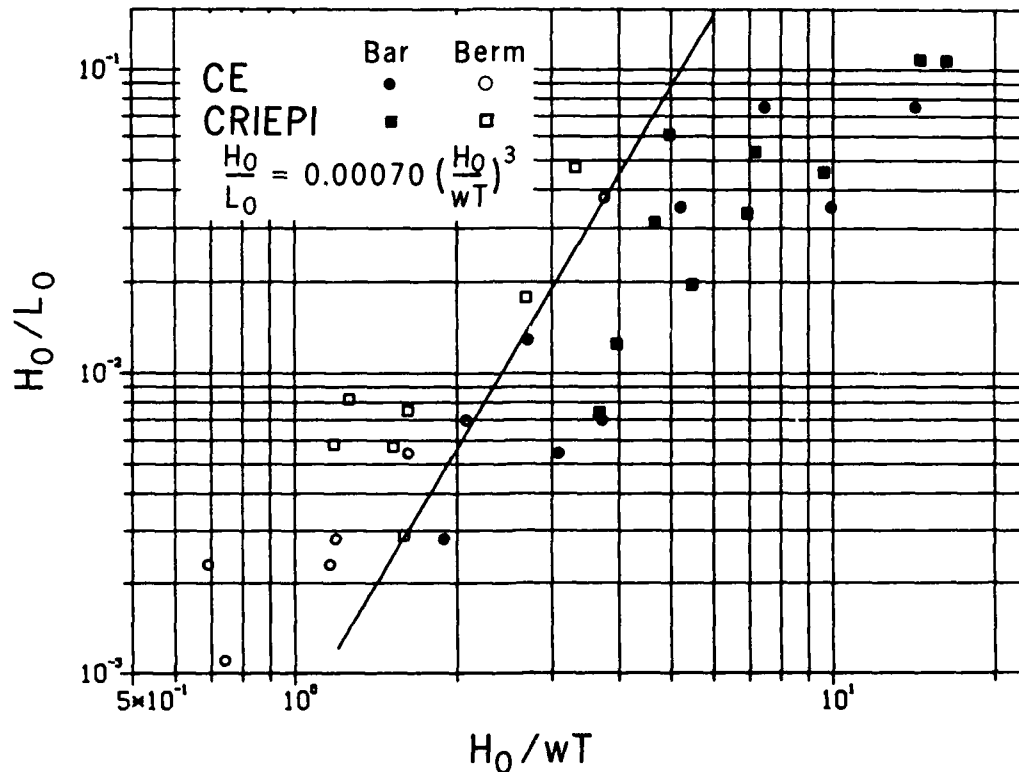


Figure 1. Criterion to distinguish bar and berm profiles

sionless fall speed and has been found to be a useful quantity for describing profile change (Dean 1973; Larson 1988; Larson, Kraus, and Sunamura 1988). Equation 3 was originally derived based on beach profile change produced by large monochromatic waves in the laboratory; recent work indicates  $H_0$  should be taken as the mean wave height in field applications (Larson, Kraus, and Sunamura, 1988).

#### Transport rate

Larson, Kraus, and Sunamura (1988) identified four zones of cross-shore sand transport, indicated in Figure 2, corresponding to properties of the local wave field: Zone I, pre-breaking zone extending seaward from the break point (BP); Zone II, breaker transition zone, between the break point and plunge point (PP); Zone III, broken wave zone; and Zone IV, swash zone. The

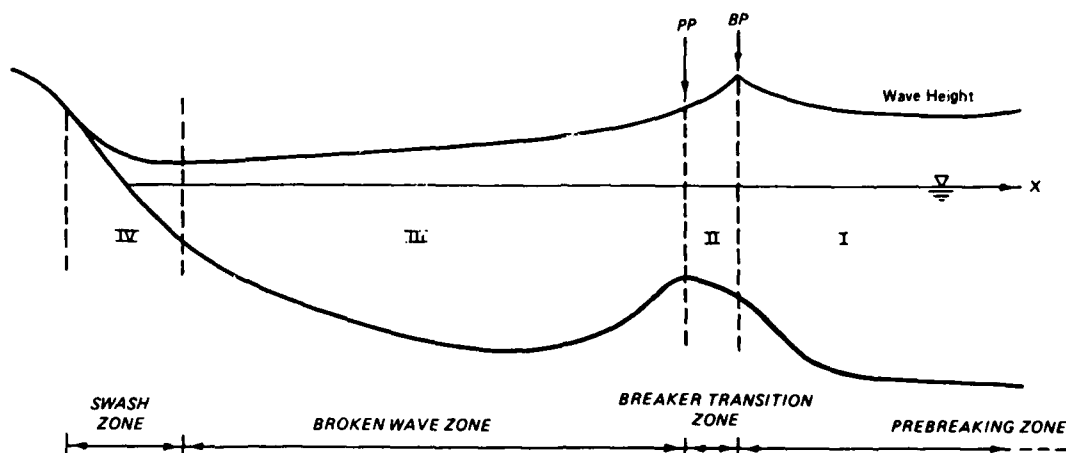


Figure 2. Definition sketch of sand transport zones

swash zone extends from an arbitrary depth, typically 0.5 m, to the limit of wave runup.

In the profile change model, the net transport rate in the broken wave zone (Zone III) determines movement of sand over the entire active profile, directly in that zone and indirectly in other zones through matching conditions at boundaries. The magnitude of the transport rate in the broken wave zone is governed by energy dissipation per unit volume in excess of an equilibrium energy dissipation as introduced by Moore (1982) and applied by Kriebel (1982, 1986) and others. A second term is included in the present model to represent the effect of beach slope. The magnitude of the transport rate  $q$  in the broken wave zone is given by

$$q = \begin{cases} K (D - D_{eq} + \frac{\epsilon}{K} \frac{dh}{dx}) & , \quad D > D_{eq} - \frac{\epsilon}{K} \frac{dh}{dx} \\ 0 & , \quad D \leq \text{above quantity} \end{cases} \quad (4)$$

where  $K$  is an empirical "transport" coefficient,  $D$  is wave energy dissipation of broken waves,  $D_{eq}$  is wave energy dissipation of a profile of equilibrium shape for the existing waves, and  $\epsilon$  is an empirical coefficient determining the strength of the bottom slope term.  $D$  is defined as:

$$D = \frac{1}{h} \frac{dF}{dx} \quad (5)$$

where  $F$  is the wave energy flux, given by shallow water linear wave theory as:

$$F = \frac{1}{8} \rho g H^2 (gh)^{1/2} \quad (6)$$

in which  $\rho$  is the density of water, and  $g$  is the acceleration of gravity.

A formal expression for  $D_{eq}$  was obtained by Dean (1977), and its magnitude has been estimated with field and laboratory data by Moore (1982). For application in numerical models of beach profile change, Moore (1982), Kriebel (1982), and Kriebel and Dean (1985) argued that if  $D > D_{eq}$ , there is excess energy dissipation and transport is directed offshore, causing the profile to erode. However, if  $D < D_{eq}$ , direct application of Equation 4 predicts a reversal in transport (neglecting the slope-dependent term), producing onshore transport. In this case, the magnitude of the onshore transport rate increases with a decrease in  $D$  and reaches a maximum for  $D = 0$ . This limit is not considered correct since a threshold energy dissipation must exist below which no significant net transport will take place. Therefore, in the present model the criterion given by Equation 3 is used to determine the transport direction, and Equation 4 is applied to calculate the magnitude of  $q$ .

Seaward of the break point, the transport rate is given by:

$$q = q_b e^{-\lambda(x-x_b)} \quad (7)$$

in which  $q_b$  is the transport rate at the break point, and  $x_b$  is the location of the break point. The spatial decay coefficient,  $\lambda$ , is found to be approximately constant with a value of  $0.11 \text{ m}^{-1}$  for accretionary conditions, but a function of the ratio of grain size and wave breaker height with a representative value of  $0.18 \text{ m}^{-1}$  for erosional conditions (Larson, 1988). In Zone II, the transport rate is described by a function of the form of Equation 7 from the plunge point to break point (by which  $q_b$  is determined), but with a value of the decay coefficient of 0.20-0.25 times that for Zone I, inferred from limited data. The transport rate at the plunge point is given by matching with the value obtained from Equation 4 at the Zone II/III interface.

Larson (1988) inferred that the transport rate distribution on the foreshore, from the shoreward side of the surf zone to the runup limit, was approximately uniform for accretionary and erosional conditions in the large wave tank experiments. A linearly varying transport rate was implemented in the model in Zone IV, constrained by avalanching (Allen 1970). Avalanching is initiated on the profile if the local slope exceeds 28 deg, and it continues until a residual angle of shearing of 18 deg is reached.

#### Profile change calculation

Changes in beach profile are calculated from the distribution of the cross-shore sand transport rate and equation of mass conservation of sand:

$$\frac{\partial h}{\partial t} = - \frac{\partial q}{\partial x} \quad (8)$$

in which  $t$  is the time. Equation 8 is numerically solved by an explicit finite-difference scheme on a uniform grid. Larson (1988) presents verification and sensitivity tests to examine model behavior under variations in empirical model parameters ( $K$ ,  $\epsilon$ ,  $D_{eq}$ ) and input data (grain size, wave height and period, and water level). The value of  $K$  applicable to field

profile change was smaller than that pertaining to the wave tank calibrations involving monochromatic waves (field value:  $K = 0.7 \cdot 10^{-6} \text{ m}^4/\text{N}$  with a range of  $0.4 \cdot 10^{-6}$  to  $0.9 \cdot 10^{-6} \text{ m}^4/\text{N}$ ). Values of  $\epsilon$  and  $D_{eq}$  used in the field tests were held the same as determined for the laboratory conditions;  $\epsilon = 0.001 \text{ m}^2/\text{sec}$ , and values of  $D_{eq}$  specified to be 25% lower than those given by Moore (1982).

## RESULTS

### Example of Model Test

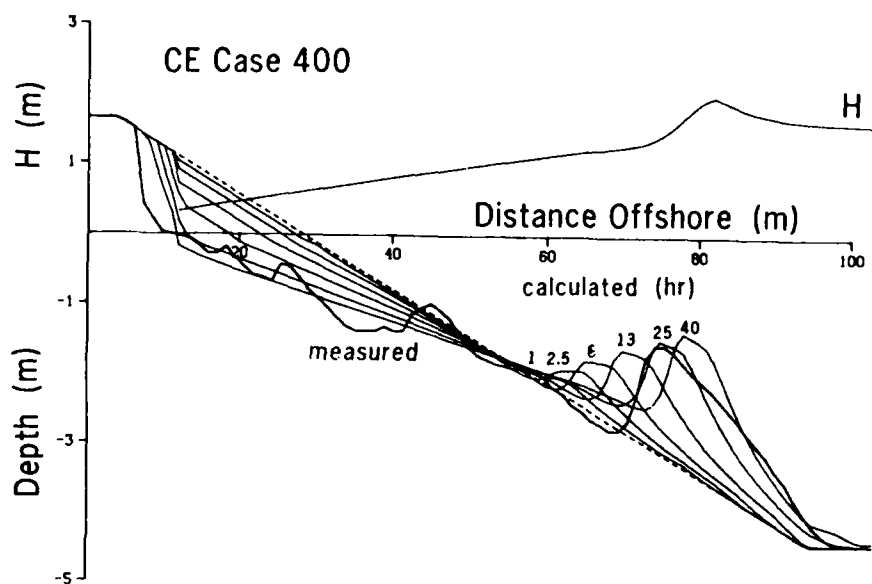
Figure 3a shows a comparison of measured and calculated profiles for CE Case 400, for which: initial slope = 1/15; grain size = 0.22 mm; wave height and period of 1.62 m and 5.6 sec in the horizontal section of the tank (depth = 4.42 m); and constant water level. These steep waves ( $H_o/L_o = 0.035$ ) cut back the foreshore to produce a vertical scarp, and a bar formed near the break point which grew and moved offshore with continued wave action. The numerical model satisfactorily reproduced the observed erosion and main bar development. Simulated bar growth was initially rapid and gradually slowed as the bar moved offshore to reach a location close to that of the observed bar at the end of the run (40 hr).

Figure 3b shows the calculated result of a hypothetical situation in which a seawall was placed at the initial still-water shoreline of Case 400. The final calculated profile for the original situation (Fig 3a) is shown as the heavier line. Development of the main breakpoint bar was quite similar for the seawall-backed beach and natural beach, whereas the profile showed greater erosion near the wall, representing local scour. In this example wave reflection at the wall was not taken into account.

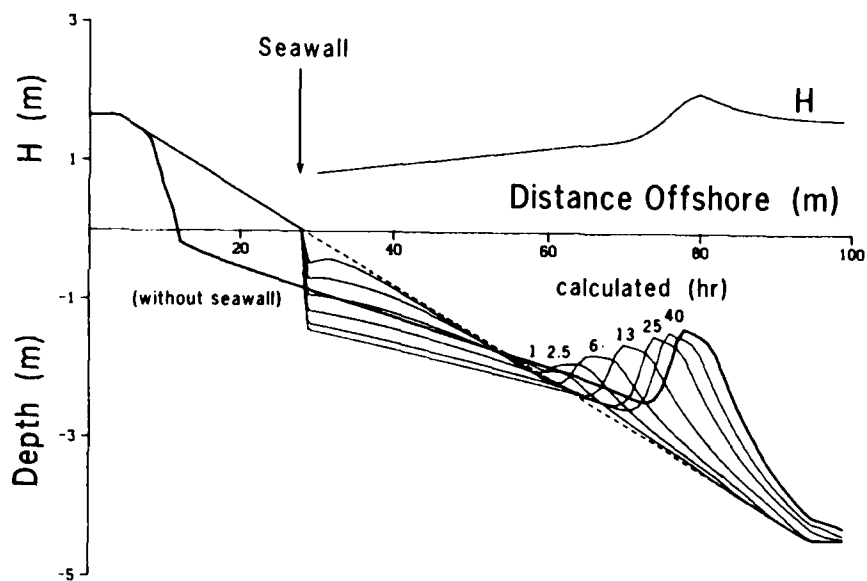
### Adjustment of Beach Fill to Wave Action

The numerical model was applied to investigate the performance of representative beach fill cross-sections. For this purpose, a design hydrograph consisting of varying waves and water level was fabricated for use in the simulations. A schematic of the time history of wave conditions and water level for the 30-day test period is given in Figure 4.

The first 21 days consist of constant waves and water level, with a wave height of 0.5 m and wave period of 8 sec. These values are meant to represent



(a) CE case 400, final measured profile and selected calculated profiles



(b) Hypothetical beach with seawall and case 400 calculation

Figure 3. Evolution of calculated foreshore erosion and bar development

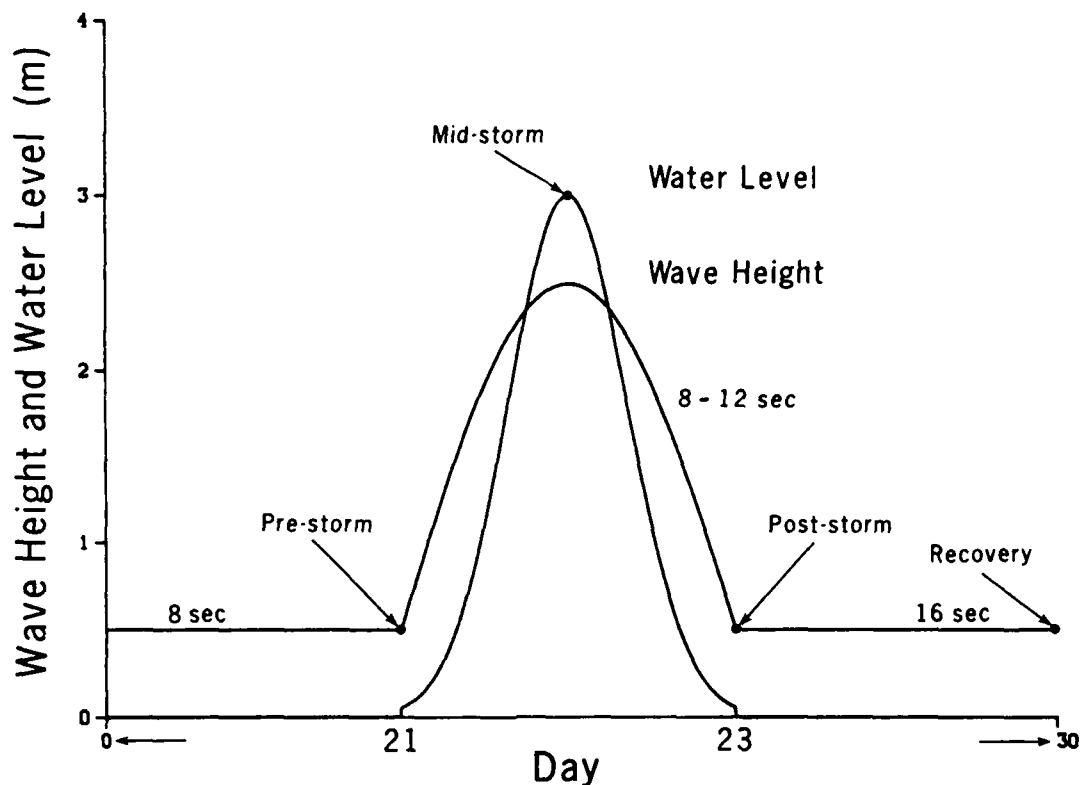


Figure 4. Time history of waves and water level

typical wave conditions on an open-ocean Atlantic coast and are used to produce a realistic equilibrium profile. Under these waves, beach change can be mildly erosional or accretionary, depending on grain size and initial profile shape. At day 21 ("Pre-storm" in Figure 4) a storm moves into the area that lasts for 3 days; the average surge water level during the storm rises and falls with Gaussian form to reach a maximum elevation of 3 m above mean sea level; the average wave height follows a sinusoid to a peak of 2.5 m at the peak of the surge ("Mid-storm"). The wave period during the 3-day event varies between 8 and 12 sec, with the 12-sec period occurring at the time of the maximum wave height. At the end of the storm ("Post-storm"), the surge vanishes and waves arrive as swell with height of 0.5 m and period of 16 sec.

In the following sections, adjustment of the profile to the 30-day design sea condition is examined with the model for three situations: an original (existing) condition, a standard fill design, and a beach advancement fill design, herein called the "Bruun beach fill." Each situation is run for two



grain sizes, 0.25 mm and 0.40 mm. For these simulations, the model grid spacing was set at 5 m and the time step at 20 min.

#### Existing condition

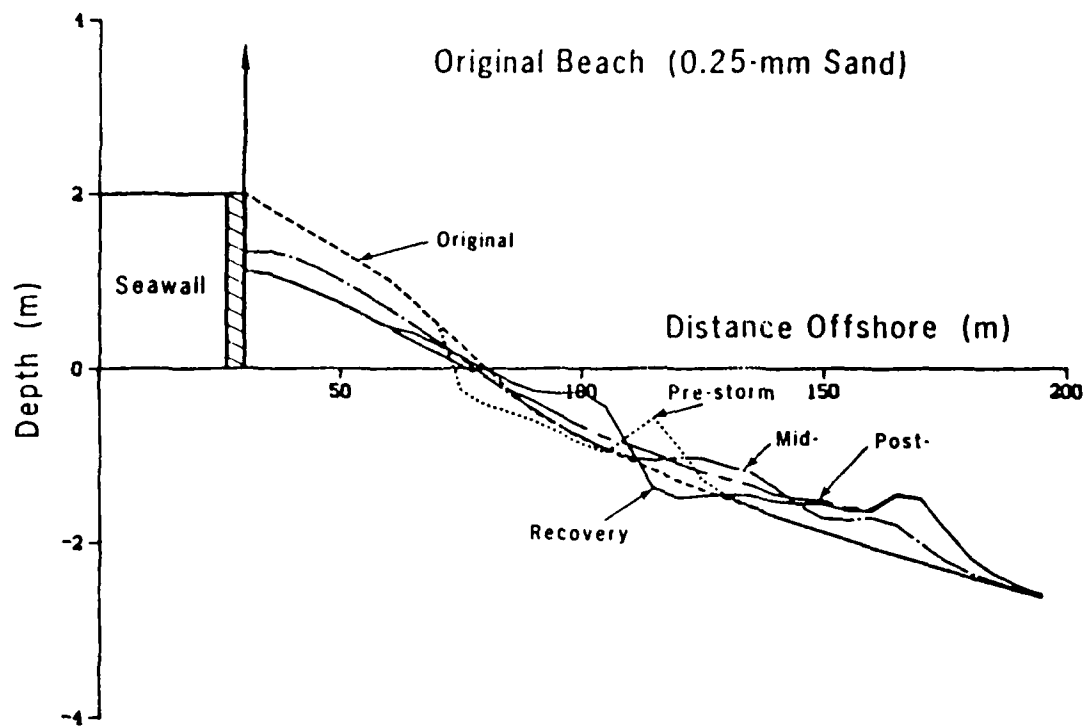
The existing condition, representing the original beach prior to nourishment activity, is shown by the dashed line in Figs. 5a and 5b. A 2-m berm is backed by an infinitely high seawall, although its crown is depicted at a 2-m elevation in this and subsequent figures. The original profile decreases linearly to the elevation of 1 m, at which point an equilibrium shape given by Equation 1 is used to extend the profile into deeper water.

Simulations showed that the 0.25-mm sand beach eroded on the foreshore, and a small bar formed with the crest located about 120 m from the base line. In contrast, the 0.40-mm sand beach accreted slightly on the foreshore, indicating that the transport was directed onshore by Equation 3. At mid-storm the berm was completely submerged and the beach eroded considerably at the seawall for both sand sizes. Offshore, a double bar developed; the inner bar was created at the beginning of the surge, whereas formation of the outer bar corresponded to the peak surge and maximum wave height. By post-storm, the berm had eroded further at the wall, slightly more so for the 0.25-mm sand beach than for the 0.40-mm sand beach, and a single large bar appeared on both profiles. The bar on the finer sand beach is broader and of lower elevation with respect to the initial profile than the coarser sand bar. It is interesting to note that the position of the shoreline at post-storm is at about the same location (approx. 75 m) for both beaches; however, sand moved further offshore on the finer sand beach, and its subaerial section is more deflated.

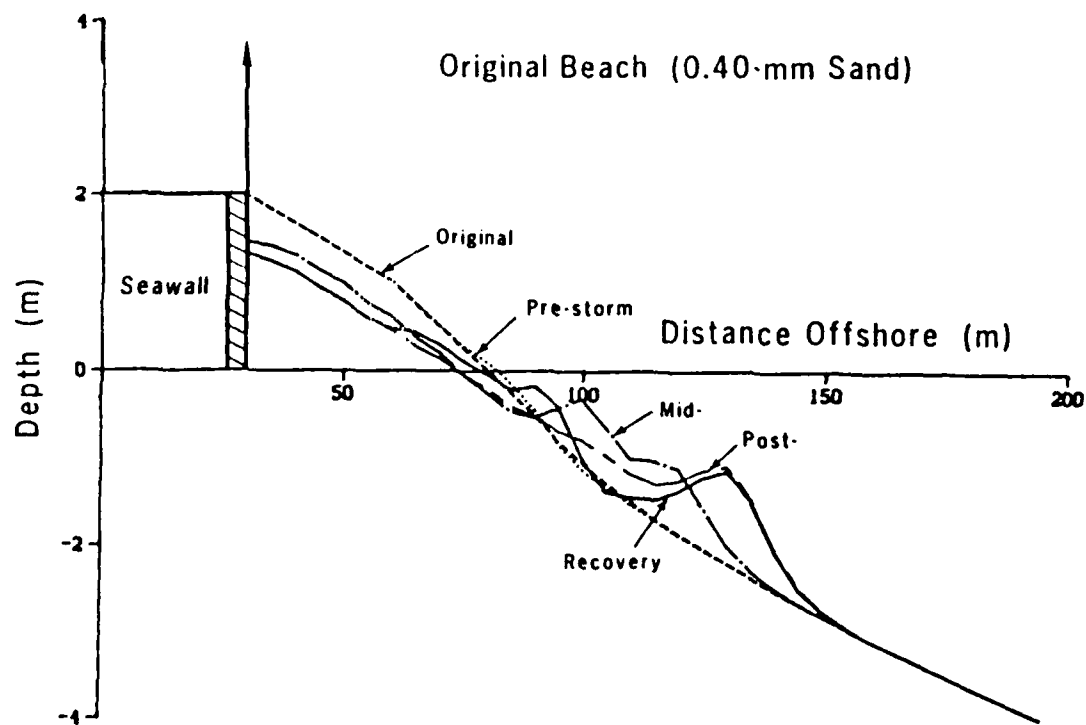
The lower-steepness waves arriving during the 7-day recovery period passed over the storm bars on both beaches, causing little change to the storm bar shape. The storm bars thus became relict bars as observed to persist in deeper water on real beaches. The recovery swell broke in shallower water, creating a small secondary bar inshore on both beaches. The inner bar was formed by onshore transport, producing a deep trough between the storm bar and inshore bar. Both beaches accreted to the limit of runup for the swell waves.

#### Artificial berm fill

Simulation results for an artificial 3-m high berm are illustrated in Figs. 6a and 6b. A beach fill of volume  $85 \text{ m}^3/\text{m}$  with the same grain size as the existing beach was placed mainly on the subaerial portion of the profile.

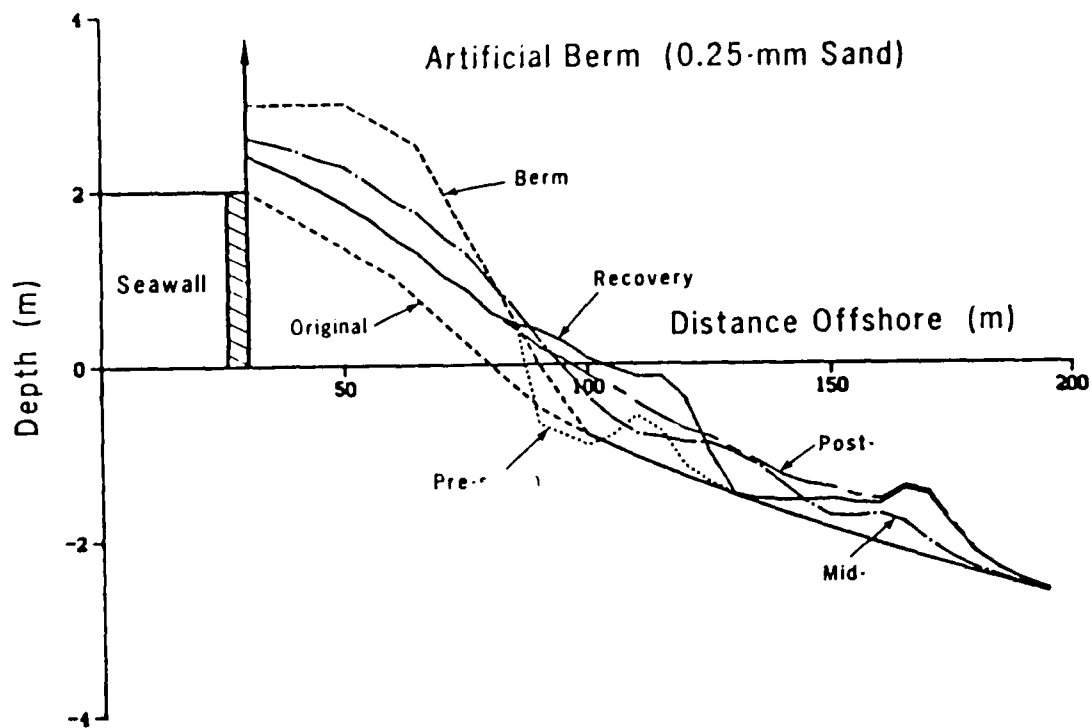


(a) 0.25-mm sand

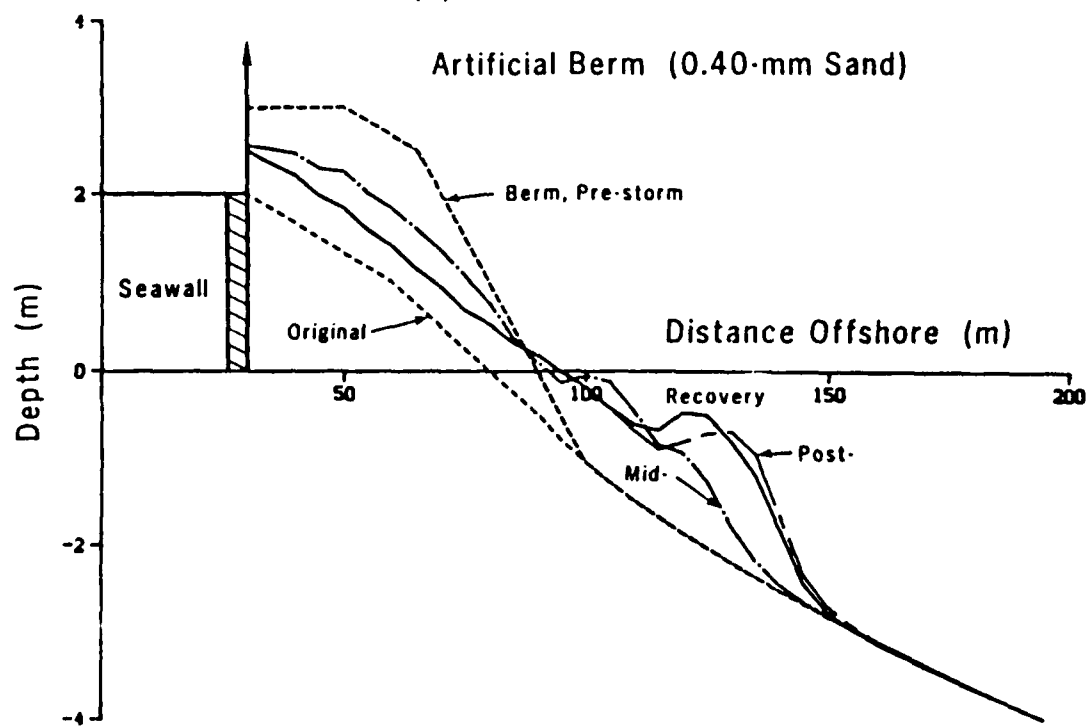


(b) 0.40-mm sand

Figure 5. Existing beach simulation



(a) 0.25-mm sand



(b) 0.40-mm sand

Figure 6. Artificial berm simulations

The fill extended horizontally from the seawall for 20 m, was tapered for several meters, then extended offshore with slope of 1/10 to meet the equilibrium profile at a depth of approximately 1 m.

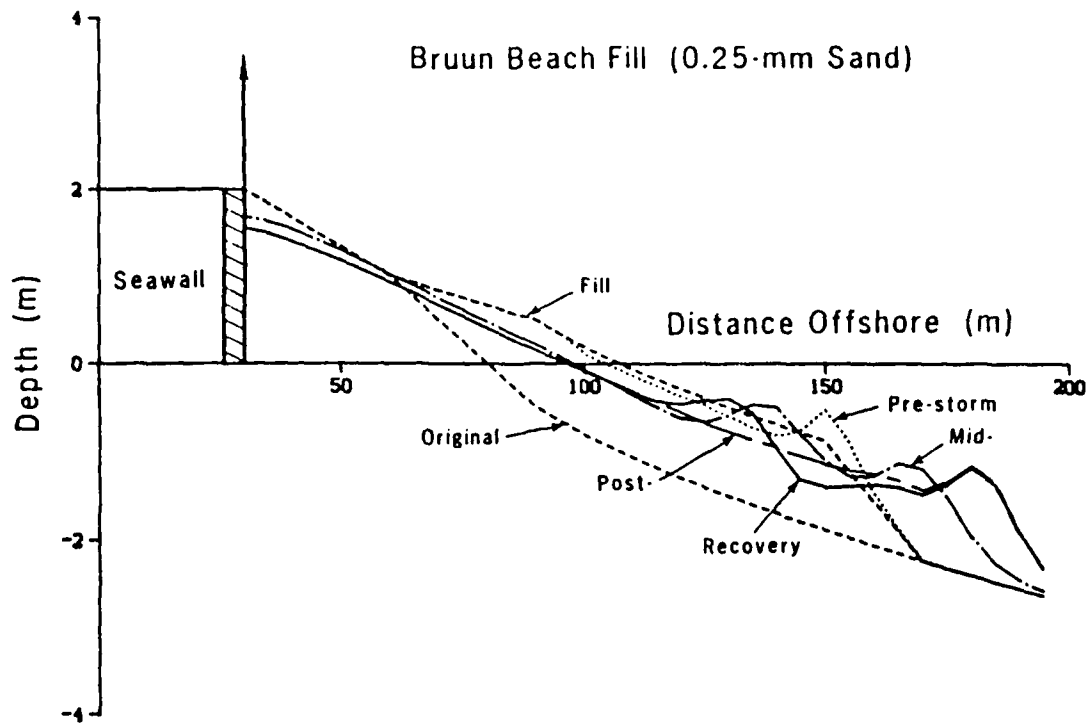
The 21 days of pre-storm waves eroded the toe of the 0.25-mm sand berm to create a small bar, whereas the 0.40-mm beach configuration was effectively stable under the mild waves. Berms for both sands were lowered during the storm, but not to the extent of the existing beach case (Figure 5); the artificial berm thus provided substantial protection, but should have been higher or wider to prevent wave action from reaching the wall for this extreme storm surge. The pre-storm bar was located slightly nearer to shore than in the existing case simulation. Post-storm bars for both sands were located in essentially the same position as for the corresponding existing beach cases.

The recovery cross-sections for both sands were considerably different than the existing beach cases because of the greater amount of sand in the system. Although significant recovery took place for the finer sand beach, sand was still trapped in the storm bar and effectively lost from the system. Eroded material for the coarser sand beach was deposited closer to shore and, during the recovery phase, the entire storm profile translated shoreward. Although no recovery of the berm took place for the coarser sand, a significant volume of material remained very near to shore.

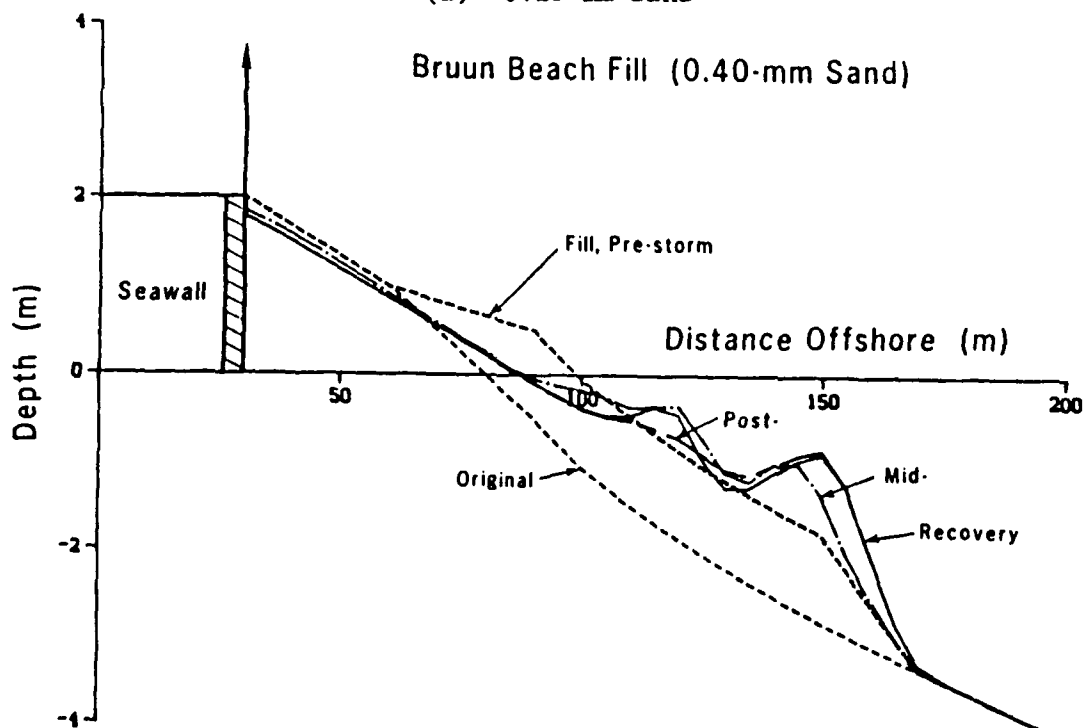
#### Bruun beach fill

The Bruun beach fill, depicted in Figs. 7a and 7b, is characterized by placement of the main portion of material over the subaqueous portion of the profile, in accordance with the philosophy that the beach can best protect itself if sand is placed over the full profile in an equilibrium shape. This type of fill has been advocated by Bruun (e.g., Bruun, 1988), who uses the term "profile nourishment" to conceptually differentiate it from nourishment of only the subaerial profile. Thus, for comparison, the same volume of fill as in the previous case ( $85 \text{ m}^3/\text{m}$ ) was placed over the profile in an equilibrium shape, as governed by Equation 1 for the particular sand grain size, and tapered at the seaward end. This fill template positioned the shoreline approximately 10 m further seaward than the artificial berm case.

Simulation results show that subaerial erosion was reduced significantly as compared to the existing beach cases, even though the entire profile was



(a) 0.25-mm sand



(b) 0.40-mm sand

Figure 7. Bruun beach fill simulations

submerged for much of the storm. A double bar system emerged again, but the location of the inner mid-storm bar for the finer material was shoreward of the pre-storm bar, caused by the rise in water level during the surge. Bar development (with respect to the initial profile) for the finer grain size was more pronounced than that for the artificial berm case, whereas the coarser grain size showed less bar formation. The Bruun fill had less overall loss of subaerial material than the artificial berm, and less material was redistributed along the profile, confirming the concept that a beach is most stable if in an equilibrium shape. However, because of its low placement elevation, the Bruun fill allowed more exposure of the wall to wave action during the surge. Berm development was absent in the recovery stage because the inner surf zone and foreshore remained near the equilibrium state to dissipate incident wave energy.

#### DISCUSSION AND CONCLUSIONS

A newly developed numerical model for simulating beach profile evolution was applied to examine the adjustment of a hypothetical existing beach profile and two fill cross-sections that might be used to nourish the beach. The model is capable of describing the growth and movement of main breakpoint bars and, to a lesser extent, corresponding berm processes. Breaking waves are assumed to be the dominant mechanism causing sand transport and profile change. The model utilizes standard engineering information as input, namely, representative deepwater wave height (mean wave height to determine net transport direction, and significant wave height to calculate cross-shore transport), peak spectral wave period, water level, and grain size (from which the fall speed is calculated). Model results are not unduly sensitive to initial profile configuration, which may not be precisely known in applications.

The model is based on relationships for cross-shore sand transport produced by breaking waves, implying that longshore sand transport can either be neglected or is uniform. Thus, caution must be exercised in applying the model and interpreting results if gradients of longshore transport exist. Longshore transport is expected to be a major contributing process to beach change over long time periods, for example, seasons to years. For such time frames, the present model should be combined with other predictive methodol-

ogies, such as a shoreline change model (e.g., Kraus 1983, Kraus, Hanson, and Harikai 1985), which compute the time rate of change of shoreline position based on estimates of wave-induced longshore sand transport.

The model produced realistic results in test calculations of profile adjustment of hypothetical nourishment projects, giving proper trends of erosion and accretion, including bar and berm formation, with respect to grain size of the beach material, wave steepness, change in water level, and initial profile configuration. The capability to reproduce beach recovery was also demonstrated to some extent. This capability is needed for long-term simulations, since beaches exhibit recovery during mild wave conditions and between storms.

Comparison of model results for the adjustment of a standard berm-type beach fill and a fill distributed over the profile in equilibrium form (the "Bruun beach fill") showed that the Bruun fill better resisted erosive action caused by steep storm waves and high surge. The Bruun fill, however, has greater potential for depletion through longshore transport, since more material is located in the active littoral zone. An artificial berm is probably more economical to construct than a Bruun fill and provides protection against inundation brought by high wave setup and surge. Where surge is a problem, combination of a high berm and a Bruun fill might provide the optimal protective design.

The economics of a fill depend on location of the material, access to the beach, type of equipment available, and the projected longevity of the fill. The present model can be used to provide guidance on the latter factor. The model can be used as a design and planning tool to assist in evaluating the efficiencies and benefits of proposed beach fill schemes, taking into account in a quantitative way such factors as fill volume and cross-section, grain size, waves, surge, and frequency of occurrence of storms.

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#### NOTATION

b subscript denoting wave breaking condition  
g acceleration of gravity  
h water depth  
o subscript denoting deepwater condition  
q cross-shore sand transport rate  
t time  
w sand fall velocity  
x distance across-shore  
A shape factor in equilibrium profile equation  
D wave energy dissipation per unit volume  
 $D_{eq}$  wave energy dissipation over a profile of equilibrium shape  
F wave energy flux  
H wave height  
K empirical transport rate coefficient  
L wavelength  
T wave period  
 $\beta$  beach slope  
 $\epsilon$  empirical coefficient in slope-dependent transport term  
 $\lambda$  empirical spatial decay coefficient in transport rate  
 $\xi$  surf similarity parameter  
 $\rho$  density of water

## PREDICTION OF BEACH FILL RESPONSE TO VARYING WAVES AND WATER LEVEL

Magnus Larson<sup>1</sup> and Nicholas C. Kraus<sup>2</sup>

### ABSTRACT

This paper describes simulations of storm-induced beach erosion performed with a newly developed numerical model of beach profile evolution. One synthetic hurricane and one synthetic extratropical storm representing typical storms with approximate 2-5 year return period are used to examine the erosion of two beach fill configurations and subsequent post-storm recovery process. Eroded volume and contour movement are evaluated as a function of storm surge, grain size, and time.

### INTRODUCTION

Over an interval of just a few hours, storms can produce serious damage and life-threatening situations on the coast by rapid erosion of the beach and upland inundation. The water motion, sand transport, and resultant rapid beach change associated with storms can usually be considered as a two-dimensional process occurring primarily in the shore-normal direction, and this assumption will be made here.

A relatively benign cross-shore counterpart of storm action is the seasonal change in beach width, with a wide beach and berm built by small, low-steepness waves in summer, and a narrow beach cut back by high, steep waves in winter. Winter waves and storms often produce one or more shore-parallel bars composed primarily of sand eroded from the beach face. Under the more gentle post-storm recovery waves and summer waves, these bars move toward shore and gradually deflate as sand is removed from them by wave action and returned to shore.

This paper presents example results of simulations of beach profile change produced by hypothetical storms calculated with a newly developed

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numerical model which has the capability to reproduce bar and berm formation and movement (Larson 1988, Larson and Kraus 1989). For this purpose, one synthetic hurricane and one synthetic extratropical storm are used to examine the response of two beach fill configurations of different grain size to storm and post-storm wave action.

#### BACKGROUND

The beach can be considered as a flexible structure that protects life and resources along the coast. Recreational beaches are themselves resources which may also have a protective function. Coastal engineering design and planning require estimation of the functioning of the protective beach in analogy to the design process for conventional engineering coastal structures such as seawalls and breakwaters.

A significant advance in the practical estimation of storm-induced dune erosion was made by Kriebel (1982, 1986) and Kriebel and Dean (1985), who used the equilibrium profile concept of Bruun (1954) and Dean (1977) to develop a relationship for the net cross-shore sediment transport rate based on energy dissipation of breaking waves in the surf zone. The resultant numerical model allows time-dependent calculation of beach and dune erosion, in which depth along the profile increases monotonically with distance offshore as governed by the incident wave energy and grain size. Scheffner (1988, 1989) embedded the "Kriebel" model in a stochastic calculation procedure to develop dune erosion-frequency of occurrence curves for evaluation of alternative dune designs. Birkemeier et al. (1987) compared available procedures and concluded the Kriebel model offered the most reliable and practical means to calculate storm-induced erosion on U.S. beaches.

Based on the success of the Kriebel model, the authors conducted a study to improve and verify the capabilities of a wave dissipation-based numerical approach for calculating storm-induced beach erosion (Larson 1988; Larson, Kraus, and Sunamura 1988, Kraus and Larson 1988, Larson and Kraus 1989). The model was established by using an extensive data set from laboratory experiments with prototype-scale waves and beaches (Larson 1988, Kraus and Larson 1988) and verified with field data. The model, called SBEACH, for Storm-induced BEach CHange, was developed with the objectives of: (1) accurate calculation of beach and dune erosion, (2) representation of bar formation and

movement, (3) representation of recovery processes (berm formation), and (4) incorporation of major water motion and beach characteristics including beach shape, grain size, and time variations of waves and water level. It was considered necessary to simulate offshore bars because they are a natural self-protective mechanism of the beach against erosion.

Technical details of the model are given in Larson (1988) and Larson and Kraus (1989). SBEACH operates by calculating the wave height at regularly spaced intervals from deep water to the shoreline. Separate empirically-based relations are then used to calculate the net cross-shore transport rate in four distinct regions along the profile: pre-breaking, breaking, post-breaking, and the swash zone. The direction of transport is determined by an empirical criterion involving the deepwater wave steepness and sediment fall speed. Basic inputs are time series of wave height, wave period, and water level; initial profile configuration; and grain size and water temperature. A finite-difference numerical scheme solves the equation for conservation of mass. The time step and discretization interval are on the order of 10 min and 1-5 m. Overall, the model satisfactorily reproduces bar formation, growth, and migration, but representation of foreshore recovery processes is incomplete. Here, the model is used to examine the effects of generic, commonly occurring storms on a beach nourishment project of two cross-sections. Kraus (1989) makes a comparison of the capabilities of various kinds of beach change numerical models.

## PROCEDURE

### Approach

Major variables controlling storm-induced beach profile change are:

- (1) Offshore bathymetry and profile shape prior to the storm.
- (2) Grain size distribution of the native beach and fill.
- (3) Surge plus tide hydrograph.
- (4) Waves (wave height, period, setup, and runup).
- (5) Fill cross-section, if any.

If long-term profile change is to be calculated, variables associated with shore-parallel processes should be included. These are the longshore sediment transport rate, fill length, and lateral boundary conditions. In the present exercise, these processes are neglected.

Two approaches may be taken for the estimation of storm impact on the beach profile; one will be called the design-storm approach and the other the storm-ensemble approach. The design storm is either a hypothetical or a historical event which produces a specified storm surge hydrograph and wave condition at the project. Surge is a water level rise caused by wind stress and atmospheric pressure variation; waves also produce a rise in mean water level at the shore called setup. The time average, on the order of an hour, of surge, wave setup, and tide is called the stage. In stage-frequency analysis, the design storm may have a certain frequency of occurrence, for example, a 100-year storm. The problem with the design storm approach for use in dune erosion modeling is that beach change is sensitive to storm duration, surge shape, and wave height and period, in addition to peak stage. The maximum water level associated with the surge of a design storm may produce less erosion than a storm of lower surge but longer duration, or than a storm of lower surge but higher waves.

The solution to the problem of the many-to-one relation between beach erosion and stage frequency is to use the storm-ensemble approach, i.e., to calculate erosion for a large number of storms (treating hurricanes and extratropical storms independently) and key the erosion to the frequency of storm occurrence (Scheffner 1988, 1989). This yields an erosion- or recession-frequency of occurrence curve. At present, the classes must be treated independently because they have different physical characteristics. For example, hurricanes are infrequent events of short-duration and high intensity, whereas extratropical storms are more frequent and usually of longer duration and lower intensity than hurricanes. The storm-ensemble approach is recommended for project design, although it requires a storm data base and is much more computationally intensive than the design storm approach. Here, the response of nourished beach profiles to a representative hurricane and an extratropical storm (northeaster) is calculated to examine predictions of the model to storms of differing waves and water levels (surge and duration). Kraus and Larson (1988) investigated profile response to a single severe hurricane. In the present work, the two storms were constructed to produce erosion resulting from a 2-5 year event for the mid-Atlantic Ocean

coast of the United States. The amount of eroded volume for such events appears to be on the order of 20-30 m<sup>3</sup>/m (Birkemeier et al. 1987).

#### **Representative Storms**

Several references and authorities were consulted in order to develop schematic storm hydrographs (surge time history), wave characteristics (height and period), and tidal variation representing those of a moderate-intensity hurricane and northeaster for use in this work. The resultant hypothetical surge hydrographs for these storms are shown in Fig. 1. The hurricane surge was developed to have a duration of approximately 12 hr, with a peak surge of 2 m, and a duration above half the peak surge (1 m) of 6 hr. The shape of the hurricane surge was generated from an inverse hyperbolic cosine squared. The surge of the northeaster has a duration of 36 hr, with a peak surge of 1 m, and a duration above half the peak surge (0.5 m) of 18 hr. The shape of the northeaster surge was generated by a cosine squared function. The peak surge of the hurricane is higher because the wind speeds in hurricanes are, on the average, greater than in northeasters.

The time history of the wave height and period assigned to the hurricane and northeaster are shown in Figs. 2 and 3, respectively. Both have peak wave heights of 5 m, which occur during the time when the respective surges are greater than half the maximum. The duration of high waves for the northeaster is thus three times that of the hurricane. Since the radius of a northeaster is typically several times greater than that of a hurricane, the fetch is longer, resulting in longer wave periods assigned to the northeaster. Wave height and period of 1 m and 7 sec were applied for approximately 6.5 days before start of the storms to mold the profiles into a realistic shape. Following the storm, the wave height and period were changed to 0.5 m and 10 sec to simulate long-period recovery swell wave conditions. A sinusoidal tide was applied with a 12-hr period and 0.5-m amplitude, and a peak in the tide occurred during the peak surge of each storm.

#### **Beach Profile Shape (fill templates)**

Following the procedure of Kraus and Larson (1988), two different beach fill cross-sections or templates were designed for exposure to the impact of the storms. One, an artificial berm, had most of the fill placed on the beach and above mean sea level (MSL). It extended horizontally for a distance of 16

m at an elevation of 3 m and then tapered with a 1:20 slope to join the original beach profile at a depth of 1.4 m (Figs. 4 and 6). The other fill strategy is termed profile nourishment (Bruun 1988), called a "Bruun fill" here, in which material was placed over the profile in approximate equal amounts from an elevation of +1 m to -2 m (Figs 5 and 7). True profile nourishment might place the fill to a greater depth, perhaps to -4 m. Because of the practical infeasibility of such a design, the Bruun fill was configured to be closer to shore. The amount of fill was the same for each template,  $140 \text{ m}^3/\text{m}$ .

Figures 4-11 pertain to a 0.20-mm sand beach, for which both the fills and the beach had the same grain size. Runs were also made for fill grain sizes in increments from 0.2 mm to 1.0 mm. In these cases, the grain size was specified as the fill size over the portion of the profile originally occupied by the fill, and 0.2 mm elsewhere. A water temperature of 20° C was specified in the model for computation of the sand fall speed.

## RESULTS

### Profile Change

Figs. 4-7 illustrate the impacts of the two storms on the beach profiles. The bold line labelled "profile without fill" gives a hypothetical dune, beach, and subaqueous equilibrium profile for reference. The solid line labelled "profile with fill" shows the fill configuration prior to storm action (at the completion of construction). The dashed "pre-storm" line shows the profile after 6.5 days of typical waves. The line with a marker represents the post-storm profile configuration, prior to the start of the recovery wave period. The line labelled "post-storm recovery" shows the profile after experiencing approximately two weeks of recovery waves.

Pre-storm: Pre-storm profiles of the berm and Bruun fills differ significantly in the inner surf zone. A steep step is produced in the berm, whereas the Bruun fill experiences gentler changes since it was placed in a near-equilibrium configuration. For both cross-sections, a small breakpoint bar formed at about the 210-220-m mark (measured from an arbitrary baseline). For all cases, material was removed from the inner surf zone and distributed along the profile beyond the depth of 2 m. Thus, regardless of the initial fill



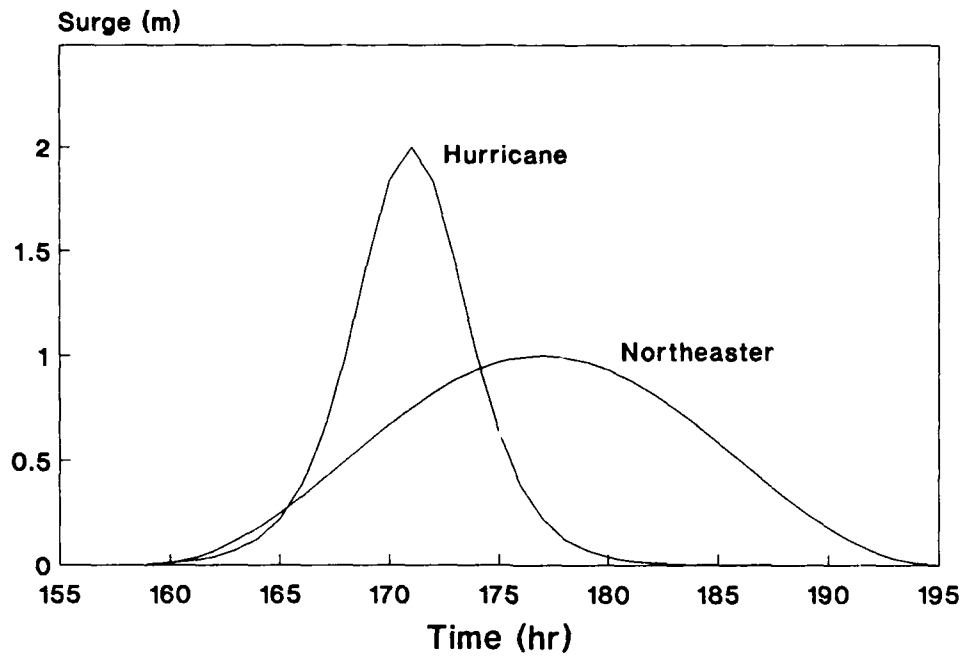


Fig. 1 Hydrographs of synthetic hurricane and northeaster

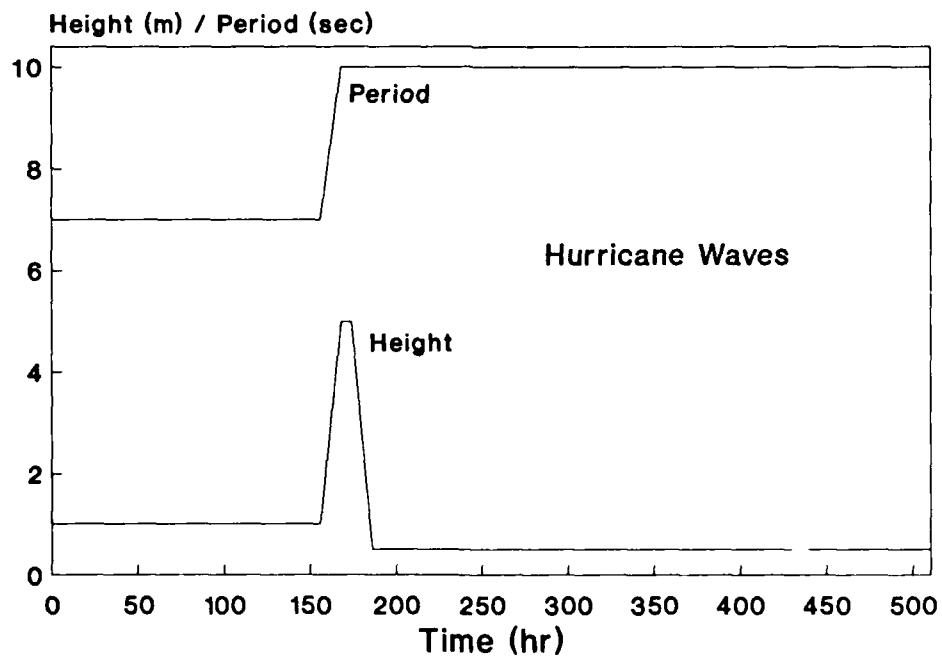


Fig. 2 Wave height and period associated with the hurricane event

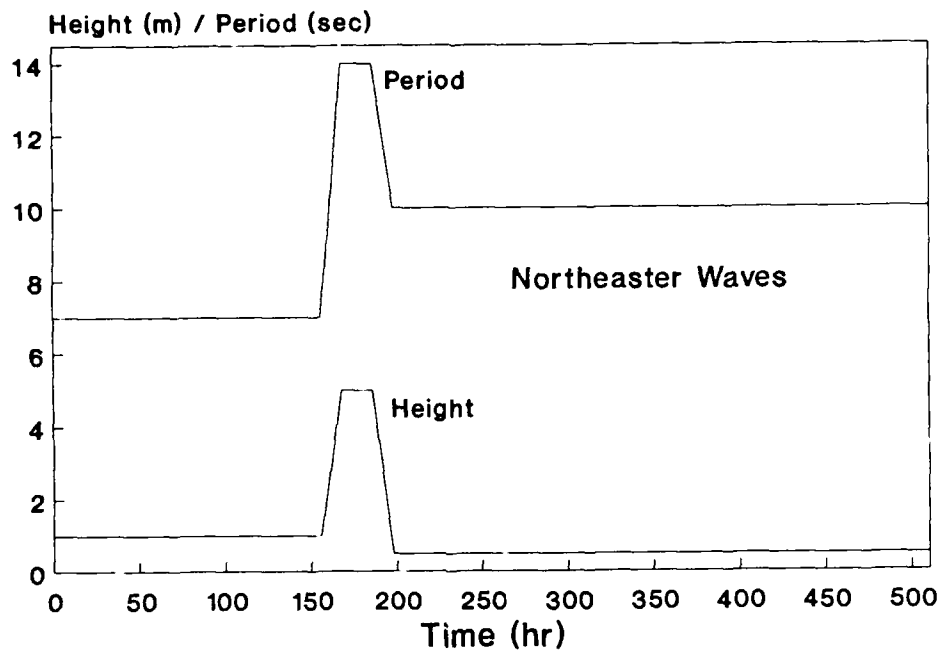


Fig. 3 Wave height and period associated with the northeaster event

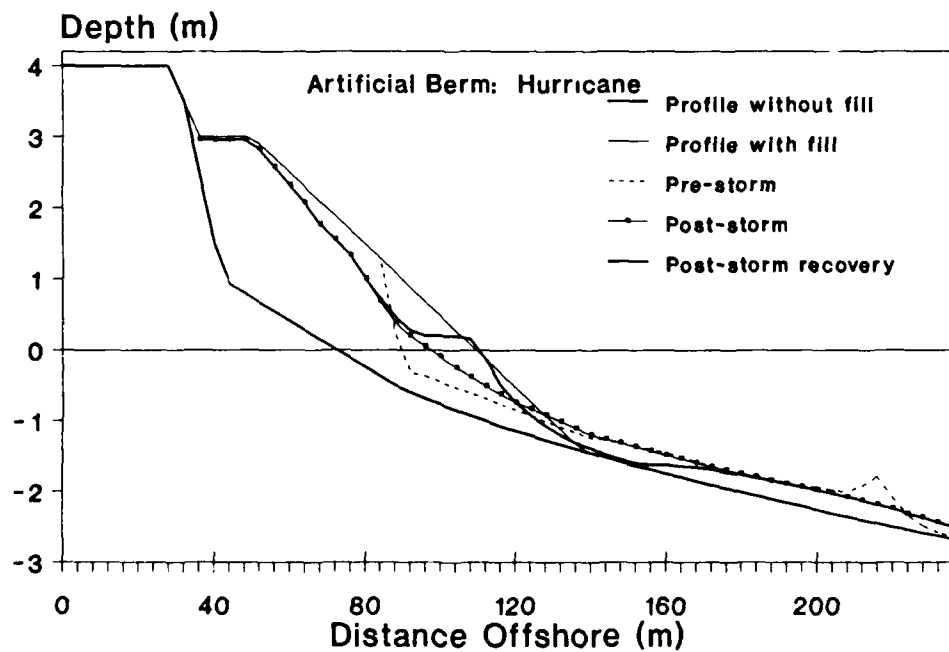


Fig. 4 Impact of the hurricane event on the artificial berm

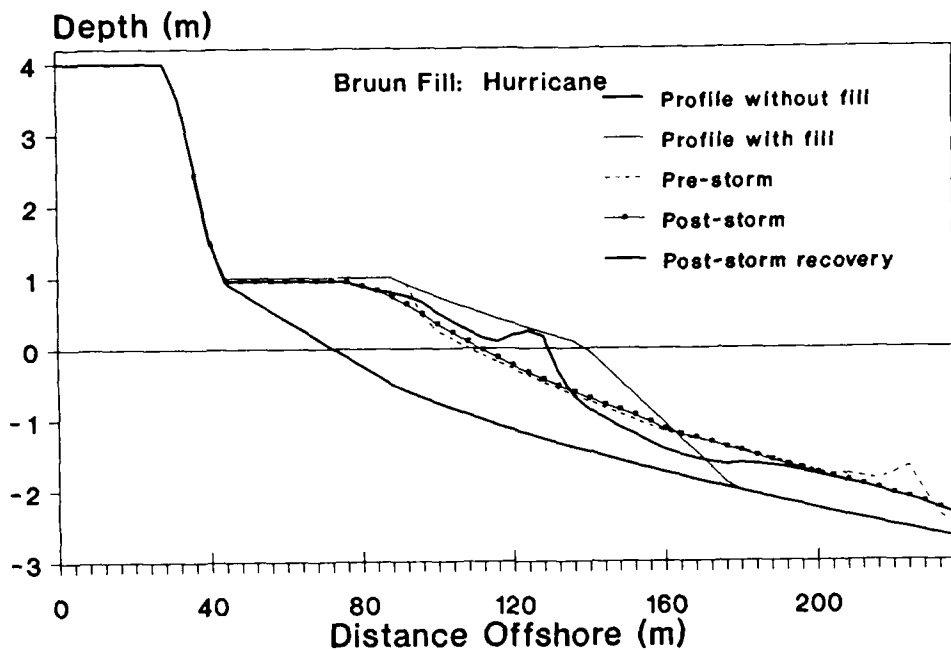


Fig. 5 Impact of the hurricane event on the Bruun beach fill

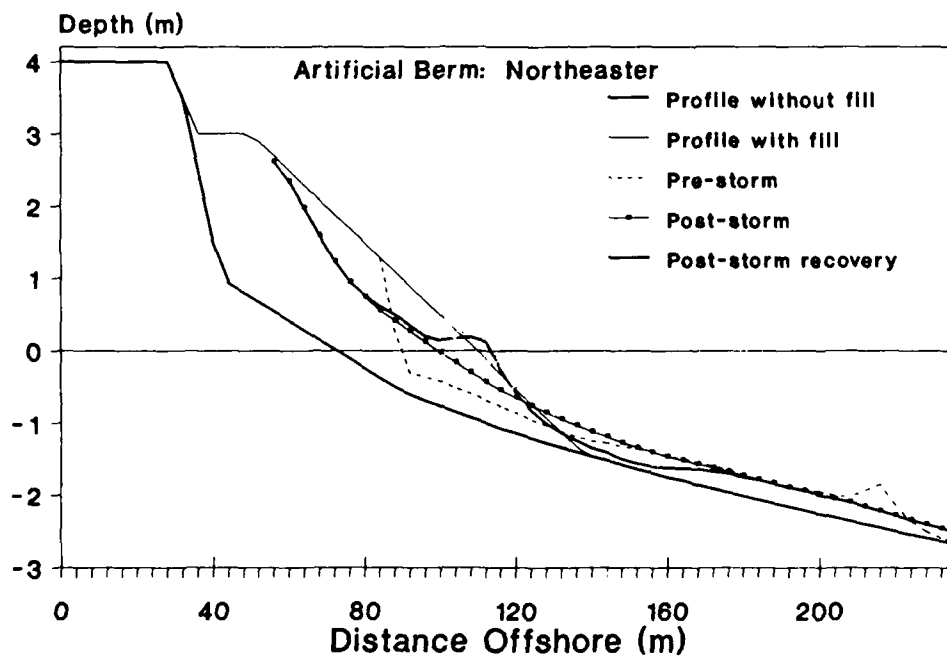


Fig. 6 Impact of the northeaster event on the artificial berm

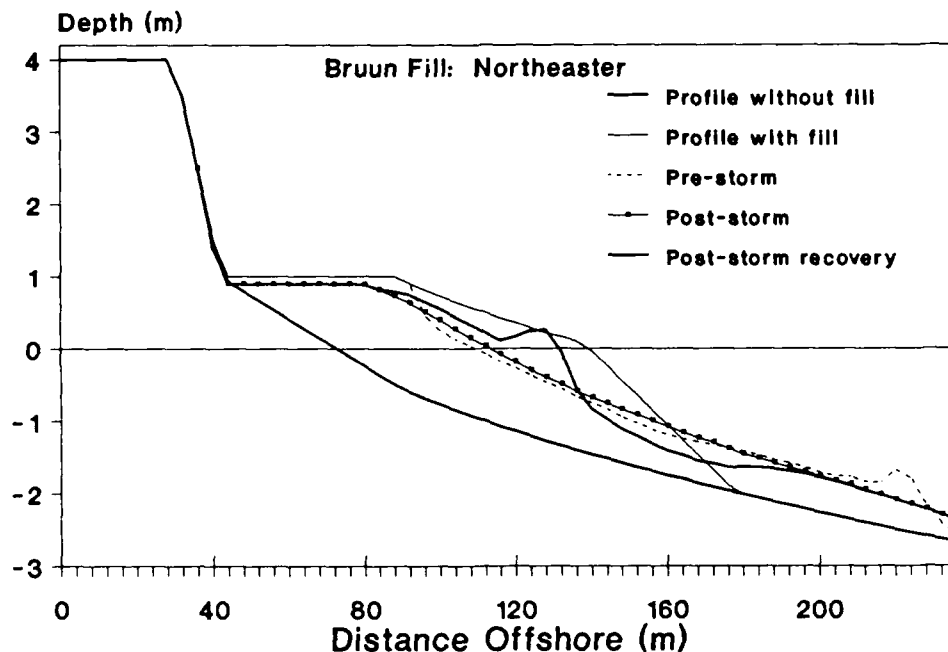


Fig. 7 Impact of the northeaster event on the Bruun beach fill

configuration, the model predicts fill material will be transported far offshore by waves in the process of molding the surf zone profile to an equilibrium shape.

Post-storm: Subaqueous sections of the post-storm profiles are very similar, being reworked by strong breaking wave action to the same equilibrium shape independent of initial profile configuration and type of storm. The shoreline position (0-depth contour, MSL) actually advanced seaward of its pre-storm location, with the material supplied from the normally subaerial portion of the profile that was inundated during the storm surge and high waves. A small bar formed at approximately 5-m depth under the high storm waves, but is not shown here to better display changes near the beach. An important outcome of the predictions is that, under the action of the particular hurricane and northeaster used, resultant profile change was very nearly the same. This demonstrates that use of one storm descriptor, for example, the maximum stage, to estimate shoreline recession or volume of eroded material can produce misleading, even dangerously erroneous results.

Post-storm recovery: In all cases, a substantial berm was created which is connected to the offshore by a broad trough. The upper foreshore of the Bruun fills experienced more accretion than the artificial berm cases. These results are consistent with the concept that the beach profile in a natural shape can best respond to changes in the incident waves.

#### **Eroded Volume and Contour Change**

In the discussion to follow, the 0-depth contour and the 1-m contour are used as references (defined with respect to MSL). It is proposed here that both the 0-m and 1-m datums be used in future studies in reporting of results storm-induced beach erosion. (Here, the 0.5-m contour was used as a substitute for the 1-m contour for the Bruun fill example because of the low relief of the fill in this particular case.)

The 0-depth contour defines the lower boundary of the subaerial beach and is a commonly used datum to define eroded volume and beach recession. However, the shoreline position often acts as a pivot point through which sand is transported; in fact, the shoreline position might even advance seaward during a storm (Birkemeier, Savage, and Leffler 1988). Thus, a second reference is needed. Although this second contour is arbitrary, the authors suggest the 1-m (3 ft) contour be used for this purpose. The advantages of reporting eroded volumes and beach recession with respect to the 1-m contour are (1) very small storms will not significantly impact this contour, so that "noise" is eliminated from the analysis, and (2) post-storm recovery will be limited at the 1-m contour, thereby avoiding a possible underestimation of eroded volume and recession. Scheffner (1988, 1989) developed dune-erosion-frequency of occurrence curves by using the maximum recession of any contour on the profile between the 0-m depth and the dune crest. Maximum recession is a good physical measure of beach erosion, but it may not be convenient for issuance of permits.

#### **Eroded volume**

Figs. 8 and 9 plot the time evolution of eroded volume above the 0- and 1-m (and 0.5-m) contours. The eroded volume above the 0-depth contour increases rapidly at the beginning of the pre-storm ("typical") wave action, describing the behavior of the fill material during initial profile adjustment. In

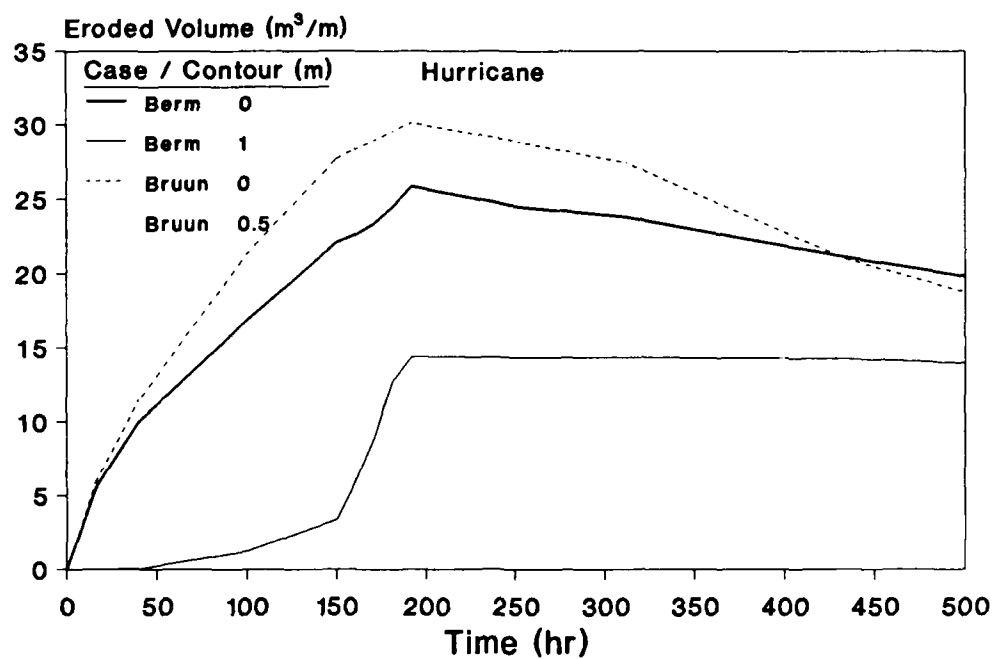


Fig. 8 Eroded volume above specific contours for the hurricane

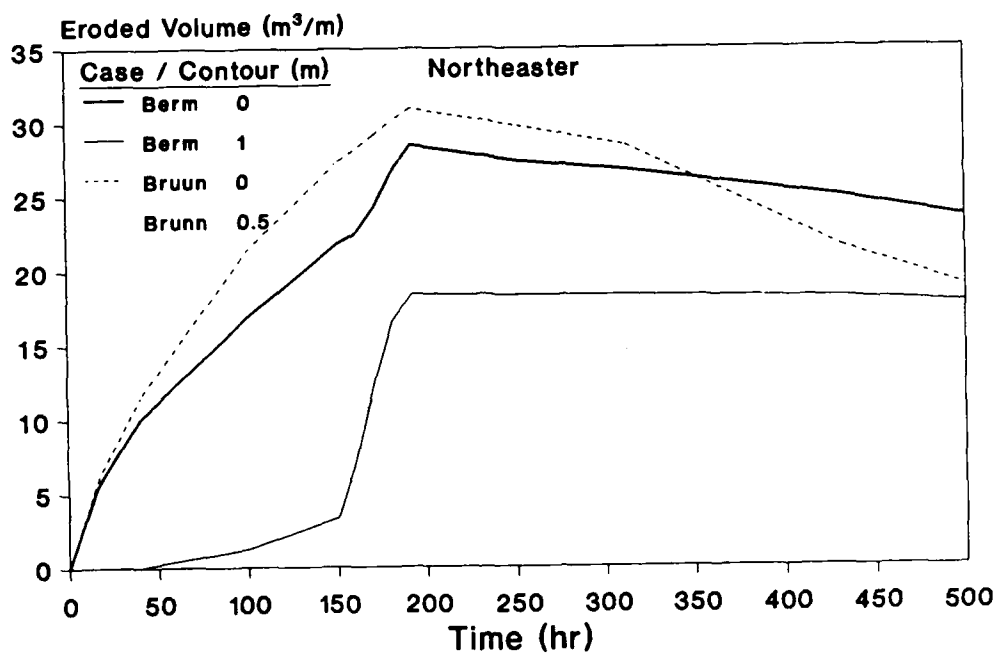


Fig. 9 Eroded volume above specific contours for the northeaster

contrast, the eroded volume above the 1-m contour shows a much less rapid increase. The Bruun fill experiences greater initial erosion during the early stage of wave action, but also greater recovery in the post-storm period. The volume of eroded material above the 0-depth contour does not show significant increase during the storms, changing only from about 22 to 27 m<sup>3</sup>/m in the case of berm erosion during the northeaster. Eroded volume above the 1-m contour abruptly increases at the start of the storms, going from about 3 to 17 m<sup>3</sup>/m in the case of the berm and northeaster. The reason why the volume of erosion above the 0-depth contour is relatively unchanged is that these moderate storms primarily remove material from the upper portion of the profile and redistribute it over the beach face, not transporting it far offshore.

The hurricane and northeaster produce about the same amount of erosion, 25-30 m<sup>3</sup>/m above the 0-depth contour and 13-16 m<sup>3</sup>/m above the 1-m contour. Eroded volumes above the 0-depth contour are comparable to those associated with 2-5 year return period hurricanes and extratropical storms impacting the mid-Atlantic coast (Birkemeier et al. 1987). The longer surge duration of the northeaster was, therefore, approximately equivalent in erosion capacity to the higher surge of the shorter duration hurricane. Time evolution of the eroded volumes above the shoreline shows an approximate exponential approach to an equilibrium value. These results are in general agreement with those obtained by Kriebel and Dean (1985), who numerically examined eroded volume and berm recession as a function of wave height, surge level, and other parameters.

#### Contours

Figs. 10 and 11 plot the time evolution of the 0-depth and 1-m contours. Decrease in magnitude of contour position indicates recession of the beach at that contour. The 0-depth contours for both fills and both storms show recession during pre-storm and storm periods, but begin to advance even before the end of the storms and prior to arrival of the recovery waves, as the surge subsides and the wave height decreases. The 1-m contour shows no recovery for the berm because the subaerial dune/berm complex is steep, whereas the 0.5-m contour for the Bruun fill does show some recovery since the gentler slope

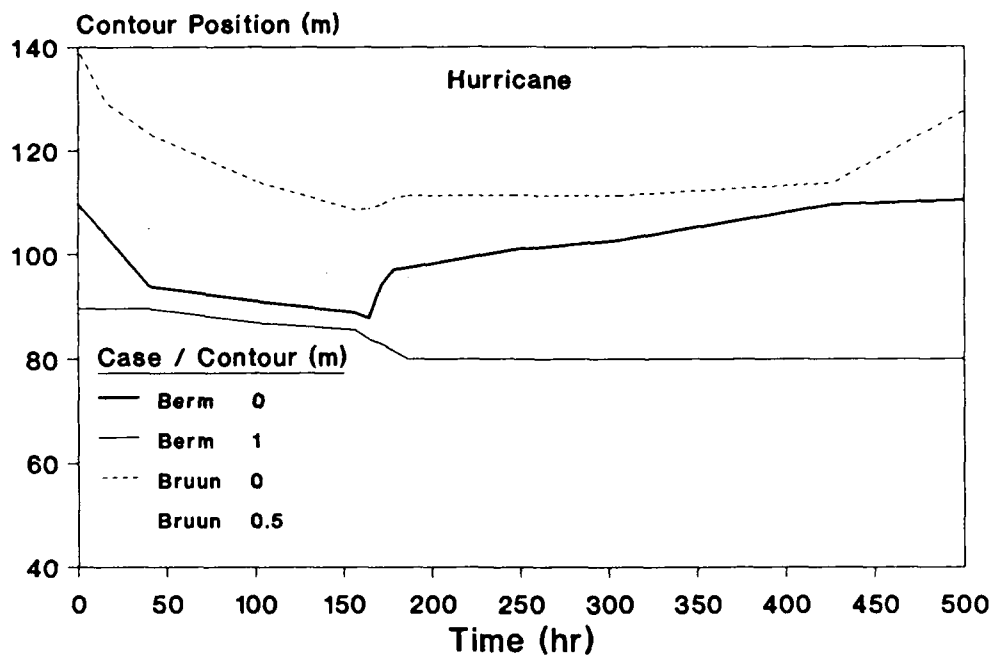


Fig. 10 Contour position for the hurricane event

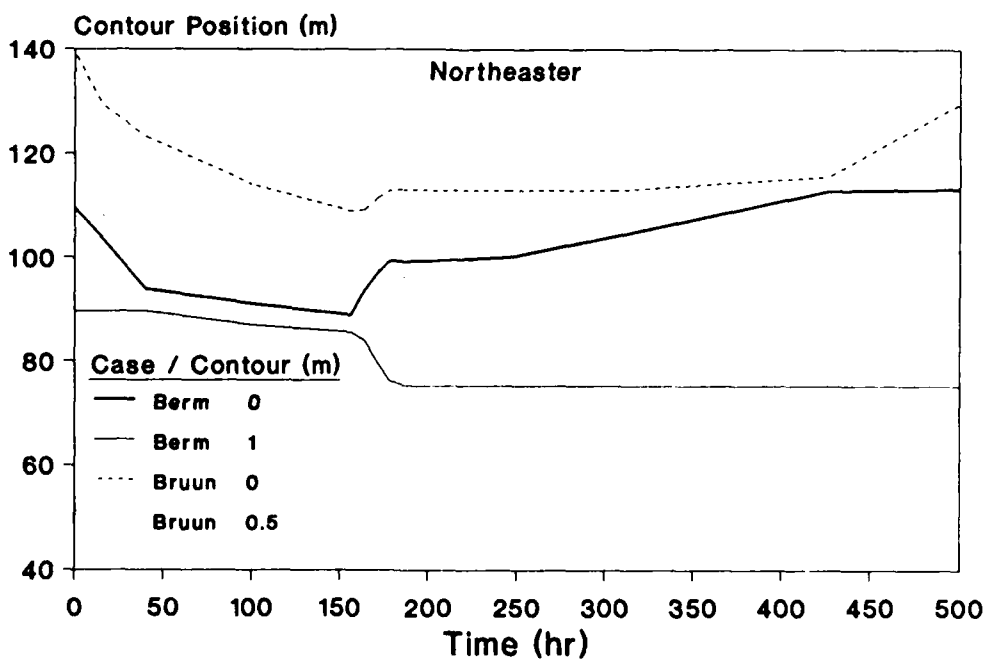


Fig. 11 Contour position for the northeaster event



allows the post-storm wave runoff to build a berm. In the model, berm formation and growth is largely controlled by the elevation reached by wave runoff (Larson 1988, Larson and Kraus 1989), which is reduced on steeper slopes.

#### Eroded Volume and Grain Size

Figs. 12 and 13 plot eroded volume at the end of the storm (prior to recovery wave action) as a function of the grain size of the fill. As previously mentioned, the native beach grain size was set at 0.2 mm, and the area in which the fill was placed was assigned the grain size of the fill. This procedure does not allow tracking of movement of the different grain sizes. However, since surf zone sediments are usually sorted with coarser material located higher on the active profile, this simple procedure is considered to provide a reasonable first approximation of the response of a natural beach of varying grain size.

Figs. 12 and 13 show a relatively steep decrease in eroded volume as grain size increases through the range of 0.2 mm to 0.4 mm, with a gentle decrease thereafter. This behavior follows from the property of the empirically determined functional dependence of the wave energy dissipation needed to generate an equilibrium profile of given grain size. This property is shown by computed dissipation rates which rise steeply in the range of 0.1 to 0.4 mm, and then increases at a lower rate with increasing grain size (Moore 1982). Since the rate of decrease in erosion is small beyond 0.4 mm, and the cost of beach fill typically increases substantially for larger size material, calculations such as those illustrated in Figs. 12 and 13 allow an evaluation to be made of initial fill and subsequent fill maintenance costs.

#### CONCLUSIONS

Main findings and recommendations from this study are:

1. Storm-induced beach and dune erosion cannot be uniquely specified through a single storm-related parameter such as the maximum stage. This result demonstrates the limited usefulness of the design storm approach.
2. The 1-m contour is a useful datum to which to refer storm-eroded volume and beach recession, in addition to the shoreline or 0-depth (MSL) datum.

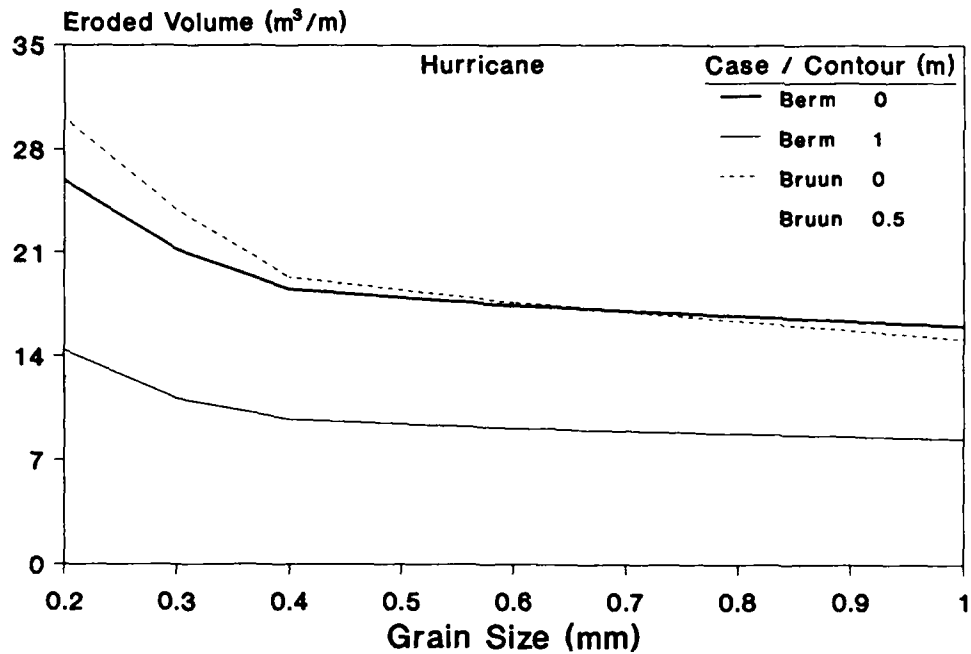


Fig. 12 Effect of grain size on eroded volume for the hurricane

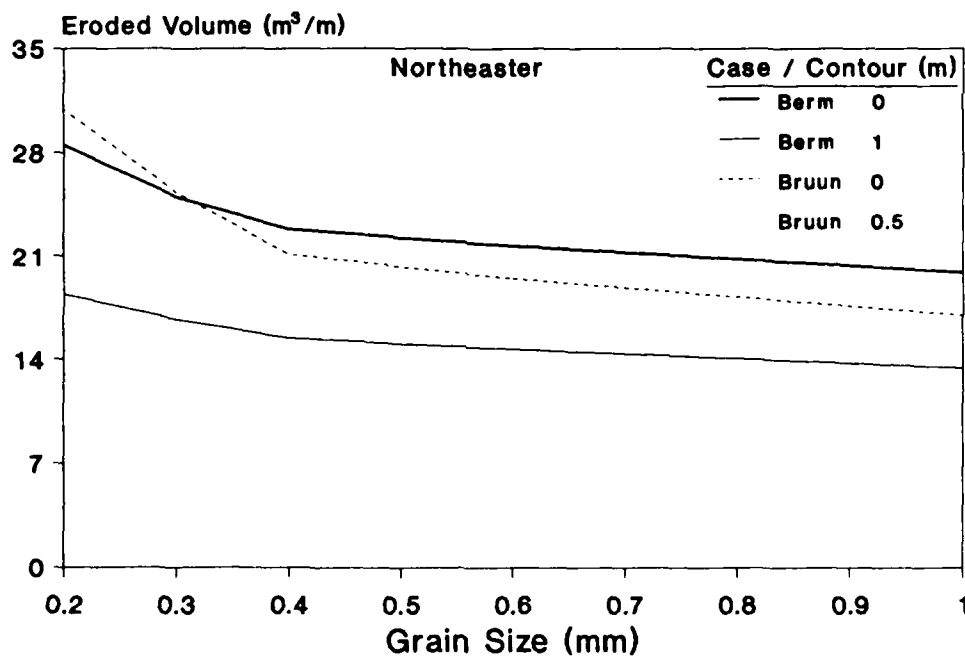


Fig. 13 Effect of grain size on eroded volume for the northeaster

3. The empirically-based numerical model used in this study and similar models provide good qualitative results for a wide range of processes associated with storm-induced beach erosion. Quantitative results are reasonable, but testing of this class of models must continue, with emphasis on field verification and model refinement. The present model describes bar formation, growth, and migration with reasonable reliability. Although not discussed here, improvements in the model are required to better represent berm growth and recovery processes (Larson 1988, Larson and Kraus 1989).
4. The model can be used to judge the relative behavior and merits of various beach fill cross-sections exposed to typical and extreme waves for time intervals on the order of days to a month (see also, Kraus and Larson 1988).
5. Fill placed on the upper beach was calculated to move offshore to relatively deep depths, in agreement with the generally inferred behavior of the movement of beach fill material.
6. The limited number of calculations performed here indicates that it may not be cost-effective to use beach fill with a median grain size much greater than 0.4 mm owing to the typically greatly increased cost of such material and the declining benefit in decreased volume of eroded material. Design alternatives for specific situations (available fill material and cross-sections) could be evaluated with the model.

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## DUNE EROSION-FREQUENCY OF STORM OCCURRENCE RELATIONSHIPS

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### ABSTRACT

This paper discusses the development and implementation of a numerical modeling methodology for making quantitative predictions of dune erosion induced by storm surge hydrographs of known frequency of occurrence. Results are in the form of site-specific maximum dune face recession versus frequency of occurrence curves. These curves can be used to assess the degree of protection afforded by an existing dune and berm complex or to evaluate the cost-effectiveness of various beach renourishment alternatives. The modeling approach was used to provide design criteria necessary for the development of a comprehensive storm erosion protection plan for locations along the north New Jersey shoreline.

### INTRODUCTION

The feasibility of any construction project is usually a function of the building cost versus the expected design life of the structure. What is the effective life of a structure before it is completely or partially destroyed by the combined action of tides and storm surge, and will the expected benefits exceed the projected costs over this lifetime? Construction costs can be accurately estimated; however, design life estimates require some means of estimating the frequency and severity of local storm events and the effect of those events on the structure.

Stage-frequency diagrams provide an estimate of the relationship between peak storm surge elevation and frequency of occurrence. These relationships are based on site-specific observations of historical storm surge data. These data provide an accurate estimate of the frequency at which damage can be expected to occur at a given elevation above mean sea level, but this procedure may not be appropriate for a structure located on or behind a protective dune line. Unless the storm surge completely overtops or breaches the dune

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crest, no appreciable damage occurs on or behind the dune. Breaching can occur due to erosion even though the peak surge level is below that of the dune crest. This category of storm-related damage can not be directly correlated with the local stage-frequency relationship since it requires a means of evaluating dune erosion as a function of storm intensity. Erosion of a dune of known cross-section and composition can be computed as a function of a specific storm event hydrograph. The erosion volume computed from this storm event can be assigned a frequency of occurrence corresponding to the peak surge level of the storm; however, this volume is not uniquely a function of the stage and return period of the storm. For example, erosion is dependent on variables such as tide elevation, storm duration, surge elevation and hydrograph shape, offshore wave conditions, wind velocity and direction, etc. (Kriebel 1982, 1984 a, b, Kraus and Larson 1988, Larson and Kraus 1989). Since events of equal surge elevation and frequency do not necessarily produce identical erosion rates, the development of a dune recession-frequency of occurrence relationship requires the simulation of an ensemble of storms with known peak surge elevations and corresponding frequencies of occurrence.

A project was initiated at the U.S. Army Engineer Waterways Experiment Station's (WES) Coastal Engineering Research Center (CERC) to develop a means of addressing the above problem in order to evaluate the effectiveness of existing and proposed dune configurations along the New Jersey coastline from Sea Bright to Manasquan (Figure 1). The approach adopted was to combine a numerical dune erosion model with an existing storm event data base. Both components described below, have been used at CERC in previous coastal studies.

#### DUNE EROSION MODEL

The dune erosion model used in this study is a modified version of the model developed by Kriebel (1982, 1984a, 1984b) and Kriebel and Dean (1985). It is a one-dimensional model based on the equilibrium offshore profile concept first postulated by Bruun (1954) and further verified by Dean (1977). This concept assumes that the offshore profile can be described by the

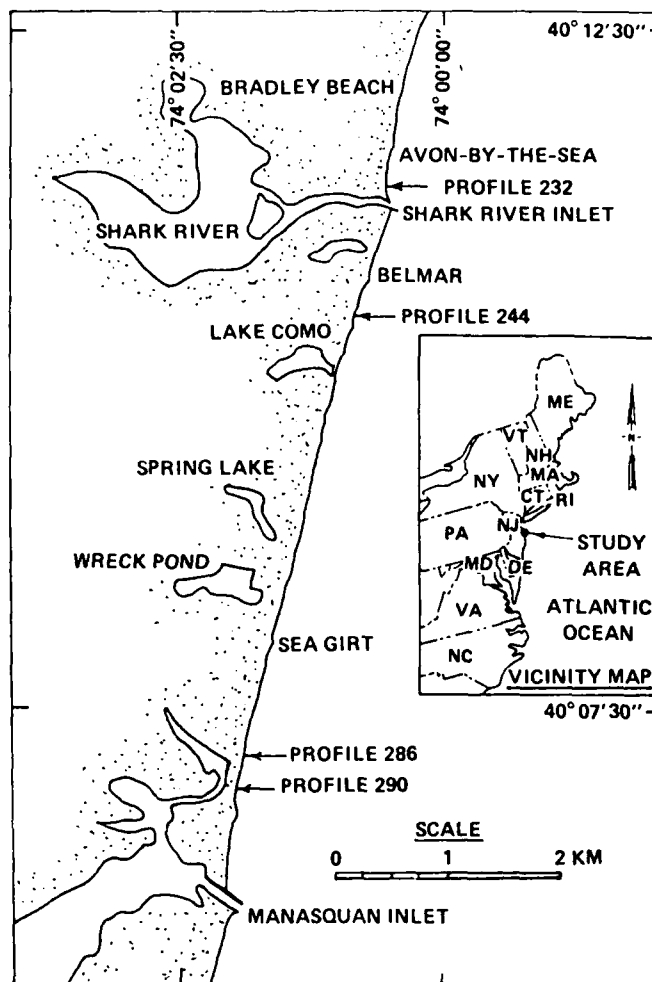


Figure 1. Project area, the New Jersey coastline from Sea Bright to Manasquan

following relationship, in which the depth  $h$  is an exponential function of the offshore distance  $x$  and an equilibrium shape coefficient  $A$ .

$$h = Ax^{2/3} \quad (1)$$

This relationship describes the active surf zone between the shoreline and the breaker line. The equation has been shown by Dean (1977) to satisfactorily represent a wide variety of beach profiles along the U.S. Atlantic and Gulf Coasts. The primary limitation of Equation 1 is that offshore features such as bars or troughs cannot be represented.



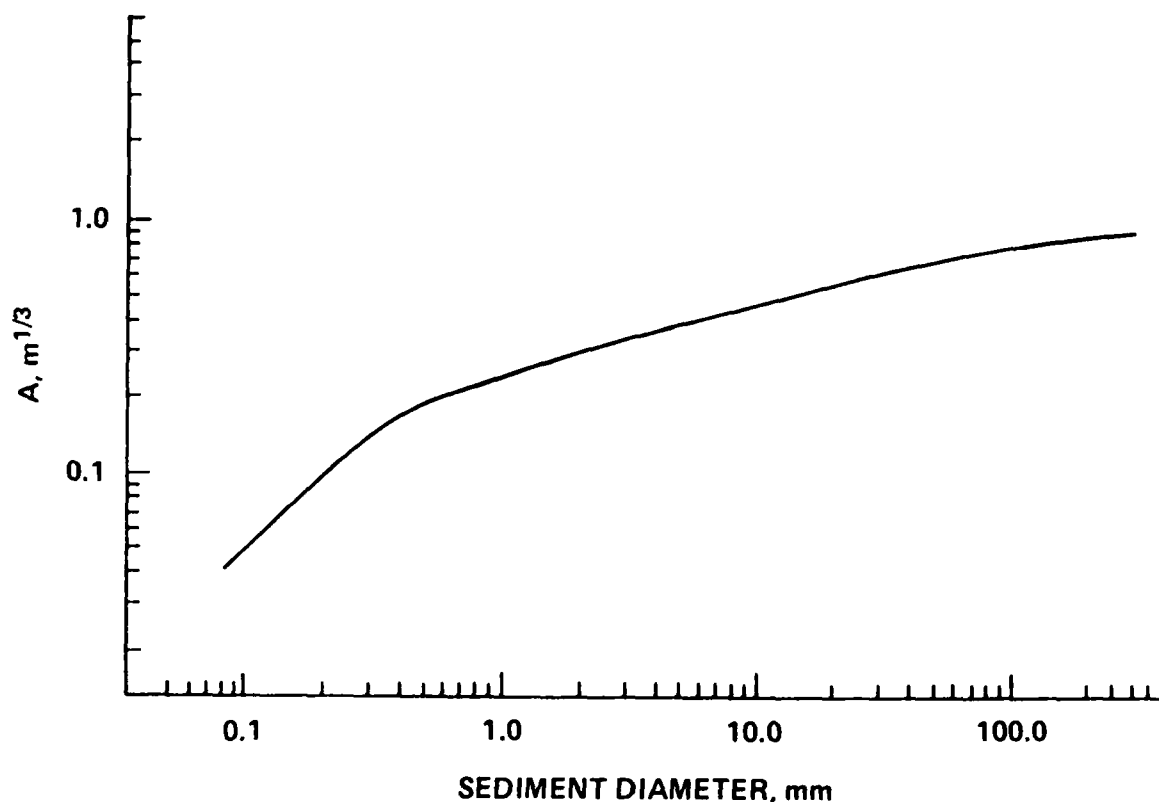


Figure 2. Shape coefficient  $A$  versus sediment diameter (from Moore 1982)

The equation is based on an assumption that the shape of the offshore profile can be related to sediments which have been sorted as a result of the dissipation of wave energy. A functional relationship has been developed between the equilibrium profile shape coefficient  $A$  and the mean grain size to demonstrate this assumption. This correlation, shown in Figure 2 (Moore 1982), indicates that larger grain sizes are reflected by larger  $A$  values. These large values describe steeper offshore profiles according to Equation 1, as would be expected of a coarse-grained beach in a high-wave energy environment.

The erosion model permits a specification of the berm and dune according to the schematic representation shown in Figure 3. The variables  $h(b)$  and  $h(d)$  refer to the height of the berm and dune, and  $M(b)$  and  $M(d)$  refer to the slope of the face of the respective berm and dune. The parameter  $W(b)$  refers to the width of an optional horizontal berm transition between the two zones.

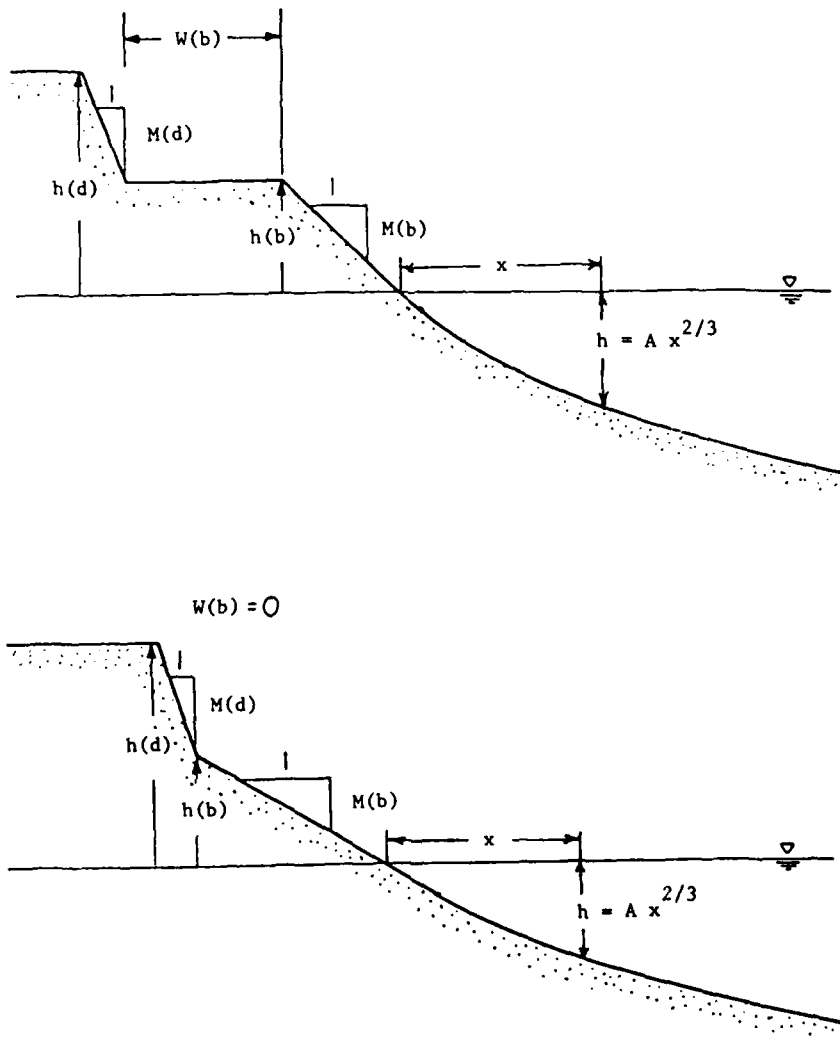


Figure 3. Schematic dune-berm profile,  
 (a) with flat berm (b) without flat  
 berm (after Kriebel 1984)

The computational approach is based on the assumption that the onshore-offshore directed transport of sediment in the surf zone is a function of the dissipation of excess wave energy per unit volume according to the relationship

$$Q_c = K(D - D_{eq}) \quad (2)$$

where  $Q_c$  represents cross-shore transport,  $K$  is an empirical transport rate parameter,  $D$  represents wave energy dissipation, and  $D_{eq}$  represents energy dissipation  $D$  for a beach in equilibrium according to the profile described by Equation 1. Wave energy dissipation can be written as a function of the gradient of the wave energy flux  $F$  per unit depth as

$$D = \frac{1}{h} \frac{dF}{dx} \quad (3)$$

If the beach is in equilibrium according to Equation 1 and all the terms in Equation 3 are expressed according to linear wave theory, the equilibrium dissipation term  $D_{eq}$  can be expressed as a function of only the equilibrium coefficient  $A$  according to the relationship:

$$D_{eq} = \frac{5}{24} \gamma \kappa^2 g^{1/2} A^{3/2} \quad (4)$$

where  $\gamma$  is the specific weight of water,  $g$  is the acceleration due to gravity, and  $\kappa$  is the breaking wave height to depth ratio (0.78). Here, the assumption is made that the surf zone is dominated by a spilling type breaker with a constant height to depth ratio.

From Equations 2, 3, and 4, it can be seen that no transport occurs for a beach in equilibrium ( $D = D_{eq}$ ), transport is in an offshore direction if the profile is more reflective (steeper) than the equilibrium profile ( $D > D_{eq}$ ), and it is onshore when the beach is more dissipative (flatter) than the equilibrium profile ( $D < D_{eq}$ ). The transport rate distribution is computed at each time step according to Equation 2 for each grid cell from the shoreline to the breaker line. A one-dimensional continuity equation,

$$\frac{dx}{dt} = - \frac{dQ_c}{dh} \quad (5)$$

is then used to compute the offshore profile response as a function of the distribution of sediment transport. In Equation 5,  $x$  represents the distance offshore to a known depth contour  $h$ , and  $t$  is time. The total

change in volume per time step is calculated from the new x-contour defined profile between the shoreline and the breaker line. Note that the time-dependent shoreline represents the intersection of the storm surge hydrograph with the face of the schematic berm or dune. This computation yields a total volume of material which is either eroded or deposited in the surf zone as a result of the input storm surge and wave field.

The key assumption in the model which relates surf zone dynamics to the berm and dune complex is that the total volume of deposition (or erosion) in the surf zone is balanced by erosion (or deposition) from the berm or dune face. If offshore deposition is indicated from Equation 5, then material is uniformly removed from the berm or dune face until the offshore volume of deposition is balanced. If onshore transport is indicated, that volume is supplied uniformly to the face of the berm. Deposition on the dune face is not permitted since dune erosion is considered to be permanent. An offshore volume of erosion or deposition is computed for each time step of the storm surge hydrograph.

The dune erosion model is capable of accurate predictions of storm induced erosion if the limitations of the model are not severely violated. Limitations include the schematic requirements of the dune and berm, and the specification of an equilibrium profile to represent the existing offshore bathymetry which may include bars or troughs. Also, the one-dimensional assumption that alongshore transport is uniform and that erosion is a function of only cross-shore transport can lead to inaccurate erosion predictions if the results are not analyzed with respect to the basic modeling assumptions. Fortunately, in many cases, the restrictions are not too severe and reasonable predictions can be obtained. The fact that many natural beach profiles can be represented by the model was demonstrated by the limited CERC verification of the model to 14 pre- and post-storm profiles (Birkemeier, et. al. 1987).

#### **STORM SURGE GENERATION**

Results of time-varying erosion of the berm and dune can be computed for any specified storm surge hydrograph. If the frequency of occurrence of that storm event is known, the erosion volume and berm or dune face recession distance can be directly related to that frequency. In order to develop this relationship, a data base of storm events with the corresponding stage-

frequency relationship is required. This data base had been generated for the study area during the conduct of a previous CERC study for Long Island, New York.

The goal of the Long Island study was to develop reliable stage-frequency relationships for specific locations in the study area. The goal was accomplished through a numerical modeling effort which computed the propagation of storm surge and tidal data from deep water into the shallow water study area. Tidal and storm surge hydrographs were generated for every grid location as a function of the specified deepwater boundary condition. Site specific stage-frequency relationships were developed by simulating the shoreward propagation of a data base of stochastically and historically generated boundary conditions.

The model used for the numerical simulations is the WES Implicit Flooding Model (WIFM) described by Butler (1978). WIFM incorporates an alternating direction implicit (ADI) finite difference algorithm to solve the depth integrated shallow water wave equations at each cell of the computational grid shown in Figure 4. The model was calibrated for tides to the primary M2 tidal constituent and verified by reproducing an observed mixed tide condition. Verification to both hurricanes and northeasters was achieved by simulating documented storms of record and comparing the computed results to recorded stage level observations. Details of the storm verification can be found in Butler and Prater (1986). The generation of the study area data base is described below.

Hurricanes and northeasters were used as the storm surge boundary conditions. Because of the basic differences in the characteristics of each storm type, the two were treated separately. Wind speed and direction data were specified for hurricanes according to the Standard Project Hurricane criteria (National Weather Service 1979). A joint probability method was used for establishing hurricane stage-frequency curves for the study area. This procedure involved the identification of the following five storm parameters: (1) central pressure deficit, (2) radius of maximum winds, (3) forward speed, (4) direction of propagation, and (5) point of landfall. Each parameter was assigned a probability based on the historical occurrence of hurricanes in the

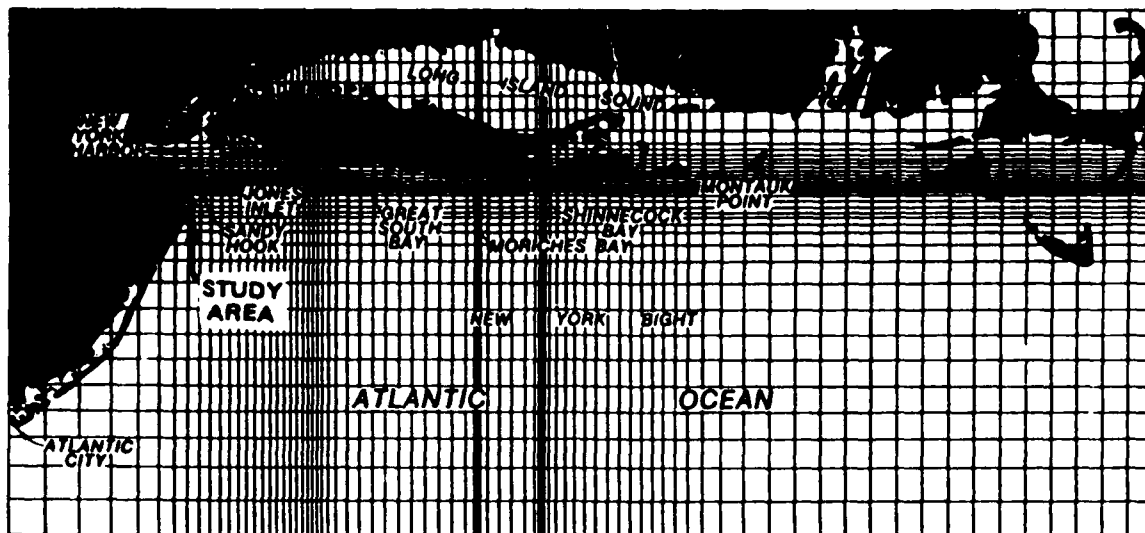


Figure 4. The Computational Grid (after Butler and Prater, 1986)

New York Bight area. Combinations of these parameters resulted in the construction of 306 hurricanes, each described by a single probability of occurrence.

Input for northeasters was based on historical wind speed, direction, and atmospheric data (Brooks and Corson, 1984). Historical records were obtained from 101 northeaster storm events for the study area. These events were identified as storms which produced at least a 0.7-m storm surge at the Sandy Hook, New Jersey, gage during the 41-year period of 1940-1980. Twenty-seven of these storms were selected as representative for the study area, each of which was assigned a historically based probability of occurrence.

The hurricane and northeaster data were used separately as the offshore boundary condition for the numerical model to compute a data base of water level time histories for each storm at each grid cell. Each of the hurricane and northeaster events was then randomly superimposed on historical tidal records at selected locations such that a total surge index was constructed which represented a combination of both tide and storm surge. This linear superposition of tide and surge neglects the nonlinear interaction of the two phenomena. This simplification is considered not to be severe for open coastal areas.

The combined surge-tide events were analyzed at the selected locations in the computational grid for which a frequency of occurrence was desired. Since the total surge elevation for each storm event at a specific area was known, a stage-frequency relationship could be computed for each storm from the ensemble of indexed storm events. The goal of the dune erosion study was to randomly select a storm of a given total surge and frequency of occurrence and subject it to a dune of a specified configuration and composition.

#### DUNE RECESSION-FREQUENCY OF OCCURRENCE RELATIONSHIPS

The two requirements for generating dune recession-frequency of occurrence relationships are: (1) a dune erosion model which computes erosion as a function of a specified storm surge hydrograph, and (2) a data base of frequency-indexed storm surge hydrographs. The mechanics of generating recession-frequency curves are described in the following example:

1. Offshore Profile. A location for which a recession-frequency diagram is to be computed must be selected. Offshore profiles should be available in order to compute an equilibrium profile coefficient according to Equation 1. If bathymetric data are not available, the coefficient can be estimated from the grain size relationship shown in Figure 2. Example profile 286, shown in Figure 5, is located just north of Manasquan Inlet, New Jersey. The equilibrium profile shape coefficient  $A$  should be chosen such that the computed profile best fits the actual profile. The fact that the computed profile does not explicitly represent bar formations is not a serious limitation of the model since the offshore computation is only intended to provide a total volume of either erosion or deposition. Since the volume is used to compute time-varying erosion of the dune and berm face, only the total magnitude of offshore erosion or deposition is of importance. A value of  $A = 0.236 \text{ ft}^{1/3}$  was computed for the example profile.
2. Schematic Dune and Berm Configuration. Each dune and berm configuration must be schematized according to the definitions shown in Figure 3. In the example, pertinent data are:  $h(b) = 8.5 \text{ ft}$ ,  $h(d) = 20.0 \text{ ft}$ ,  $M(b) = 0.131$ ,  $M(d) = 0.110$ , and  $W(b) = 65.0 \text{ ft}$ .
3. Stage-Frequency Diagram. A stage-frequency relationship is required for the selected area. These diagrams can usually be obtained from existing literature. If a relationship is not available, an interpolation between gage sites for which the stage-frequency diagrams are available may be acceptable. If the study area is not conducive to interpolation, a relationship should be constructed based on historical data. For the present application, stage-frequency diagrams computed for the Long Island study were used. Diagrams for both hurricanes and northeasters, corresponding to three tide gage locations, are shown in Figures 6 and 7.

4. **Storm Surge Data.** Storm surge hydrographs with peak levels corresponding to finite stages along the stage-frequency curve are required as input for the erosion model. For the present study, an ensemble of 120 northeasters, corresponding to five separate simulations of discrete total surge elevations (storm surge plus tide) from 5.0 ft to 9.6 ft in 0.2-ft increments, and 275 hurricanes, with total surges from 4.0 ft to 14.8 ft at 0.2-ft increments, were randomly selected from the data base of 600,000 hurricane and 18,000 northeaster indexed surge-tide events.
5. **Dune and Berm Erosion Simulations.** Each storm event of the ensemble of storm hydrographs was input to the dune erosion model. The maximum amount of recession computed for any contour line between the dune crest and mean sea level during the entire storm simulation was selected as an indicator of maximum dune erosion. Although recovery is experienced on the berm, this maximum recession was selected to be a realistic indicator of maximum damage. Figures 8 and 9 represent the computed scatter diagrams for the dune recession-frequency of occurrence relationships for hurricanes and northeasters. A design curve was defined from the upper envelope of data points. The hurricane and northeaster curves were then combined into a single design curve. Both the individual component curves and the design maximum recession-frequency of occurrence curves are shown on Figure 10.

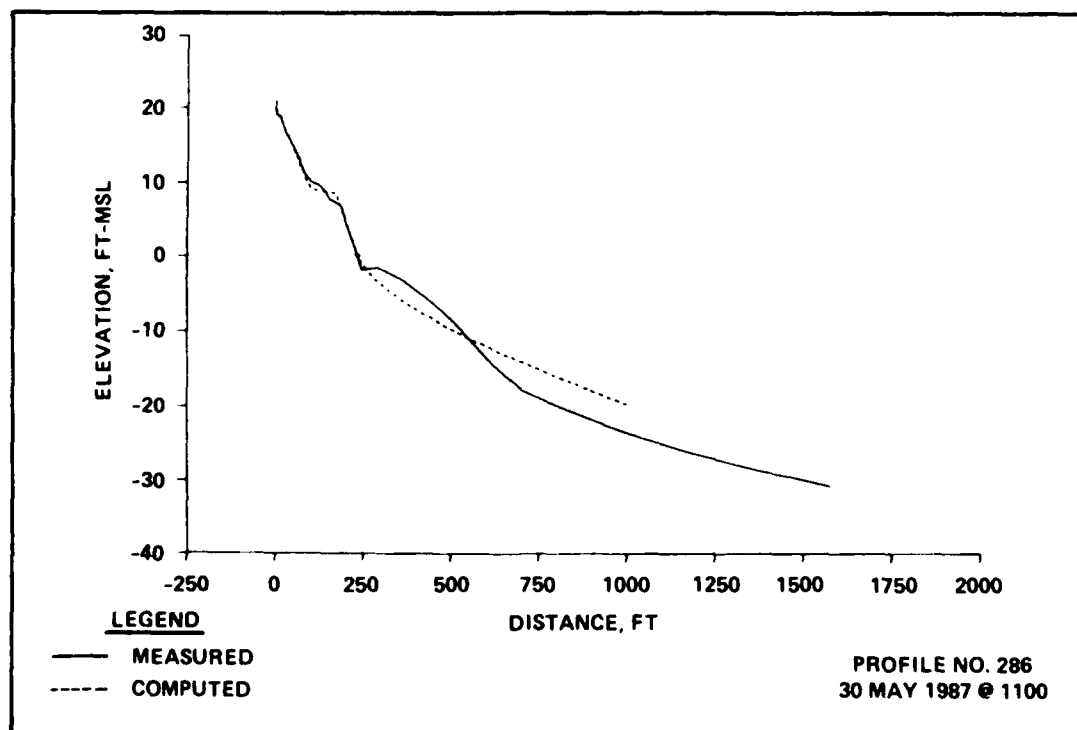


Figure 5. Cross-section of profile 286



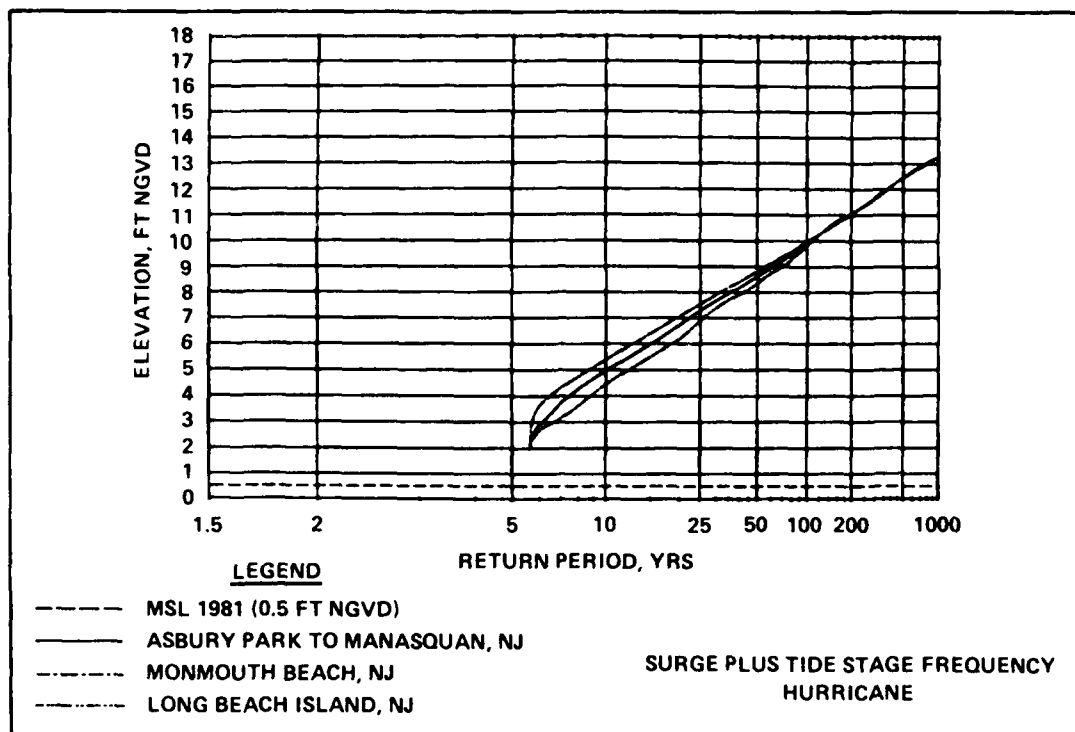


Figure 6. Hurricane stage-frequency diagram for the study area

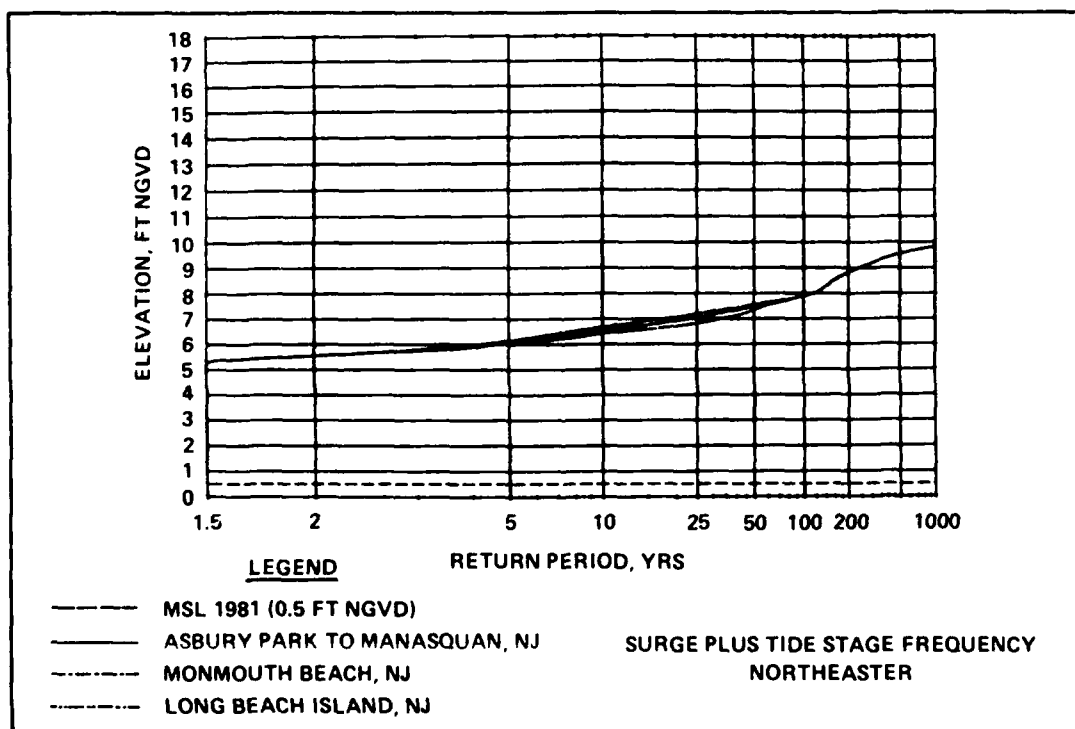


Figure 7. Northeastern stage-frequency diagram for the study area

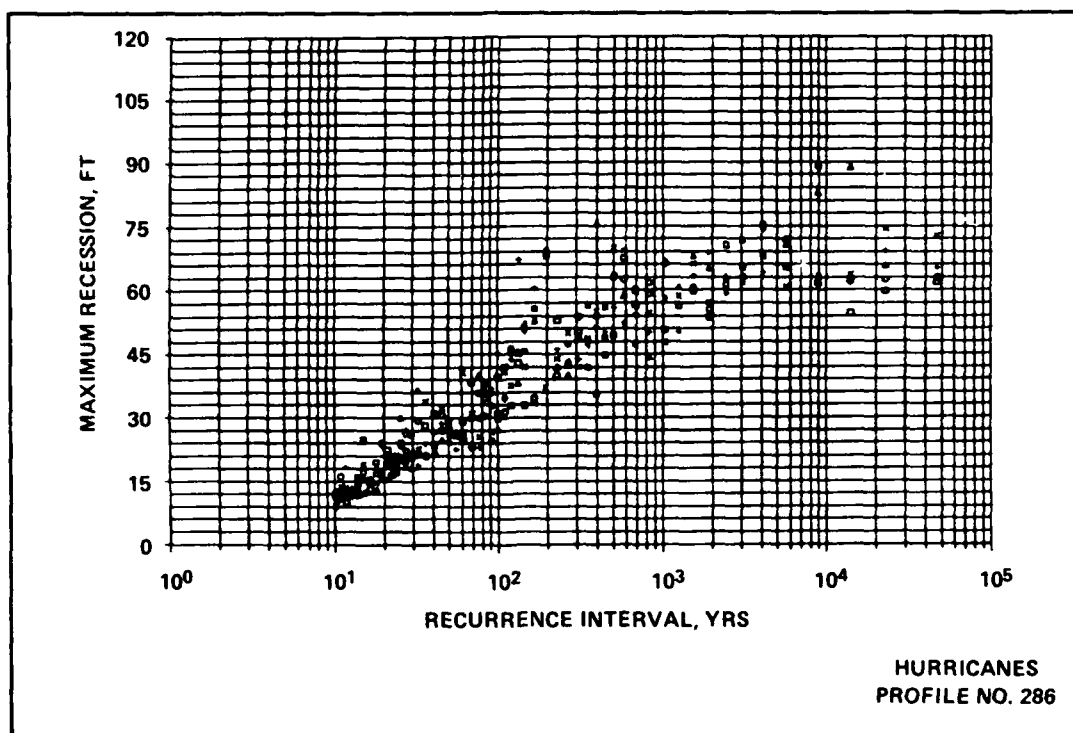


Figure 8. Hurricane recession-recurrence interval diagram for the study area

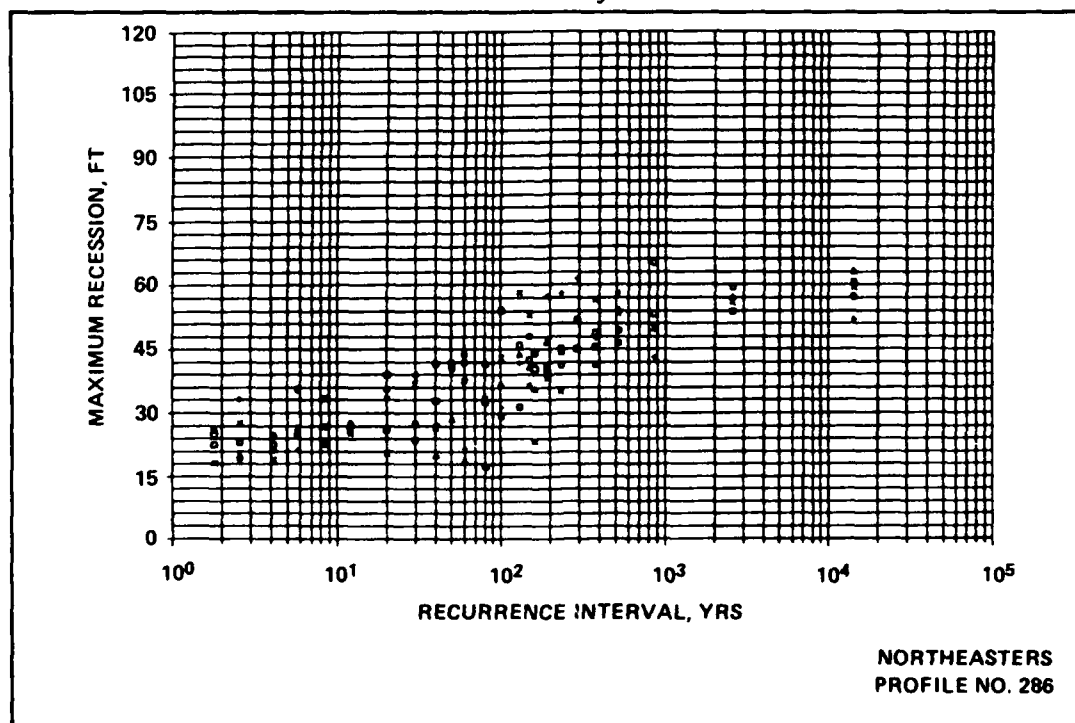


Figure 9. Northeastern recession-recurrence interval diagram of the study area

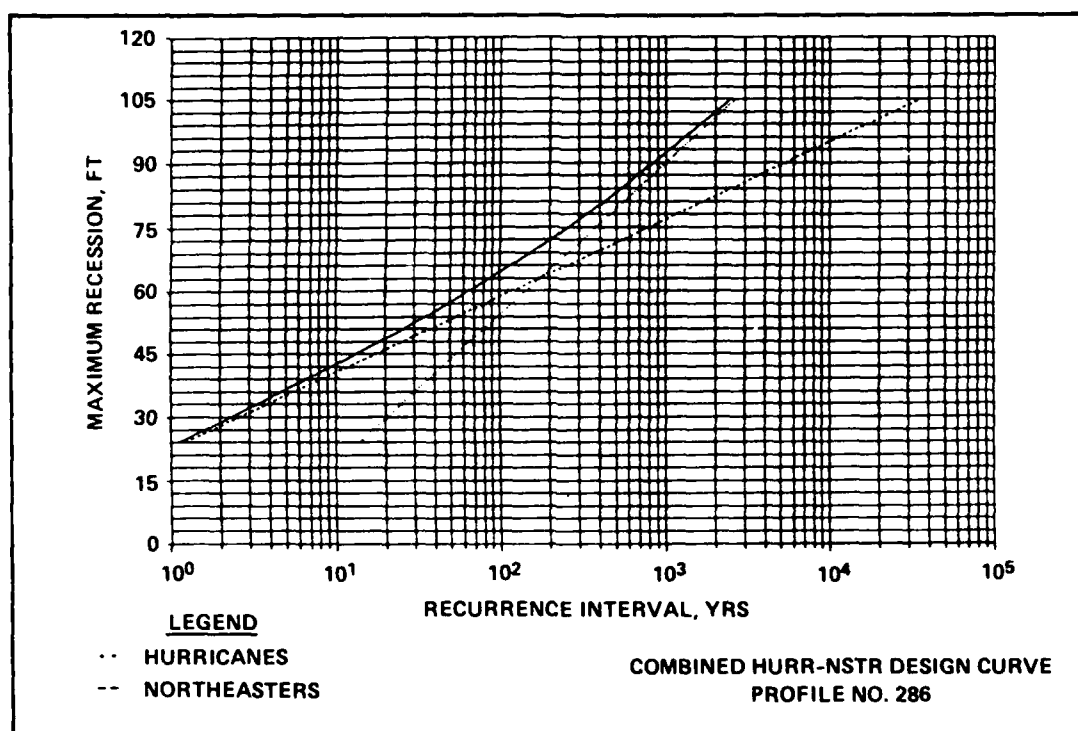


Figure 10. Combined hurricane-northeaster recession-recurrence interval design curve for the study area

## CONCLUSIONS

A systematic means of predicting time-dependent erosion of dunes as a function of storm events of known frequency of occurrence was developed by combining the existing technologies of stochastic stage-frequency analysis and a single event, one-dimensional dune erosion model. The technique employed produces dune recession-frequency of occurrence relationships which can be effectively used for estimating the design life of structures protected by the presence of a berm and dune.

## ACKNOWLEDGEMENTS

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